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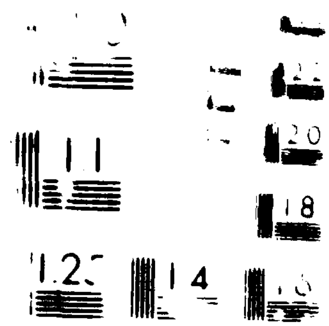

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US Army Corps of Engineers

AD-A189 077

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TECHNIQUES FOR IMPROVING ENERGY EFFICIENCY AT WATER SUPPLY PUMPING STATIONS

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M. Walski
A. Sharp

DEPARTMENT OF THE ARMY

orps of Engineers

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This report presents an application of a methodology for reducing the amount of energy used for pumping in treated water distribution systems. The methodology can be divided into three steps: (a) field testing of pumps to ensure that they are performing to specifications, (b) determining the best combination of pumps to achieve a given discharge, and (c) determining the optimal discharges (or tank water levels).

The methods presented in the report are applied to pumping stations serving the water distribution system for Washington, DC, and the vicinity. They indicate that energy savings of approximately 5 percent ($90,000 per year for the two pressure zones investigated) can be realized without adversely affecting service. The methods can be used in developing rough rules for pump operation or for real-time control of pumping.

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PREFACE

This report describes the application of methods presented in Engineer Technical Letter (ETL) "Energy Efficiency at Water Supply Pumping Stations" to the Washington, DC, and vicinity water system. Both the ETL and this report were prepared under the Water System Operation, Maintenance and Rehabilitation Work Unit (CWIS 31794) of the Water Supply and Conservation Research Program. The technical monitors for this program in the Office, Chief of Engineers, were Mr. James Ballif (DAEN-ECE-B) and Mr. Robert Daniel (DAEN-CWP-D).

The work was conducted at the US Army Engineer (USAE) Waterways Experiment Station (WES), Vicksburg, Miss., and the University of Kentucky (UK), Civil Engineering Department. The report was written by Dr. Lindell E. Ormsbee, assistant professor of civil engineering at UK, working with WES under an Intergovernmental Personnel Act agreement; Dr. Thomas M. Walski, a research civil engineer with the Water Resources Engineering Group (WREG) of the Environmental Engineering Division (EED), Environmental Laboratory (EL), WES; and Messrs. Donald V. Chase and Wayne W. Sharp, UK students employed by WES under the contract student program. Mr. Anthony C. Gibson of the WREG assisted in field data collection.

Work done with the Washington Aqueduct Division (WAD) of the USAE District, Baltimore, was performed under the purview of Mr. Harry C. Ways, Chief, WAD; Mr. Perry Costas, Assistant Chief, WAD; and Mr. Douglas B. Pickering, Chief, Plant Operations Branch, WAD.

The report was reviewed by Mr. M. John Cullinane of the Water Supply and Waste Treatment Group, EED, and Dr. Keith W. Little of the Research Triangle Institute, Research Triangle Park, N. C. The report was edited by Ms. Jessica S. Ruff of the WES Information Technology Laboratory.

The study was conducted under the supervision of Mr. F. Douglas Shields, Jr., Acting Chief, WREG: Dr. Raymond L. Montgomery, Chief, EED; and Dr. John Harrison, Chief, EL.

Commander and Director of WES was COL Dwayne G. Lee, CE. Technical Director was Dr. Robert W. Whalin.

This report should be cited as follows:

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CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
gallons (US liquid)	3.785412	cubic decimetres
horsepower (550 foot-pounds (force) per second)	745.6999	watts
inches	2.54	centimetres
miles (US statute)	1.609347	kilometres
pounds (force) per square inch	6.894757	kilopascals

TECHNIQUES FOR IMPROVING ENERGY EFFICIENCY AT WATER SUPPLY PUMPING STATIONS

PART I: INTRODUCTION

Background

1. Significant changes in the cost and availability of energy in the United States have made energy management an important priority. Almost [illegible] percent of the electricity consumed in the United States is used by municipal water utilities (Brailey and Jacobs 1980). In conventional surface water systems, pumping may comprise up to 90 percent of the total energy budget. In ground-water systems where there is no treatment other than chlorination, pumping may account for more than 95 percent of the energy requirement (Reheis and Griffin 1984). From these percentages, it is clear that the major effort in energy conservation programs for water systems should be spent on improving pumping operation efficiency.

2. In recent years, several authors have shown that significant energy savings can be obtained by improving the operation policies associated with treated water pump systems (Trainer and Clopton 1976, Lizardos and Amato 1978, and Reid 1980). Attempts to improve pump operation efficiency may focus on three different operation problems: inefficient pumps, inefficient pump combinations, and inefficient pump scheduling. Each problem is discussed in detail in the following paragraphs.

3. For single-pump operation, improved efficiency may be obtained by reduction of the pumping head, reduction of the volume of water pumped, or an increase in the pumping efficiency (Patton and Horsly 1980). In order to evaluate the efficiency of an existing pump audits driver, the pump unit must normally be field tested (Gros 1976). An evaluation of the energy consumption and performance of a pump can also provide valuable information on its general condition. From this information, a rational decision can be made on the cost-effectiveness of repairing or replacing a low-efficiency pump (Aldworth 1983). Hodnik and Frye (1983) provided some methods for identifying inefficient pumps and incorporating efficiency into pump selection.

4. In addition to evaluating the efficiency of individual pumps, it is also important to evaluate the efficiency of multiple-pump combinations. For

pumping stations with multiple pumps, energy savings may be identified by examination of the overall efficiencies associated with operation of different combinations of pumps. Although different combinations of similar pumps may deliver the same approximate flow rate for a given head, some combinations may be less costly because of differences in pump efficiencies. In some cases, the efficiency of a pump when running alone can be significantly different than when it runs in conjuction with other pumps. Since the majority of water plants do not have written guidelines for operators to follow for obtaining optimal performance from their pumps (i.e., least-cost energy), pump selection at any time may be inefficient in terms of energy usage even though the operator's selection of pumps meets the flow and pressure demands (Reheis and Griffin 1984).

5. Although the evaluation of pump efficiencies may lead to a reduction in total energy usage charges, the reduction of time-of-day energy charges requires a modification of operation procedures over time (Chao 1979, Lackowitz and Petretti 1983). Real-time operation of a pumping system involves two different decision-making processes: development of an operating rule curve (tank level versus time of day) and implementation of the rule curve. For pumping systems that operate under essentially constant conditions, the rule curve may remain the same from day to day. For systems whose conditions vary greatly from day to day, the rule curve may have to be updated daily or even hourly (Shamir 1985).

6. Clingenpeel (1983) described the results of some studies to reduce energy consumption by taking advantage of off-peak power rates. Brunzell (1983) described how storage could be used to take advantage of electric rates and reduce demand charges.

7. Procedures for generating operation rule curves have been developed by DeMoyer and Horwitz (1975), Sterling and Coulbeck (1975a, b), and Sabet and Helweg (1985). The majority of these procedures were based on dynamic programming and were developed for relatively small, single-reservoir systems. Attempts at developing rule curves for more complex systems have been presented by Donachie et al. (1976), Damelin and Shamir (1976), Carpentier and Cohen (1984), and Shamir (1985).

Purpose

8. Although several authors have addressed individual aspects of pump operation efficiency, there remains a need for a comprehensive methodology for use in evaluating and improving the overall operating efficiencies of treated water pumping systems. The purpose of this study is to provide such a methodology. As part of this program, the District of Columbia (DC) and vicinity water system was selected for use as a case study for the application of the developed methodology.

9. The proposed methodology has been divided into three basic components. The first component concerns the field testing and evaluation of individual pump units. A review of fundamental concepts of pumping system design and operation is provided in Enclosure 1 of Appendix A. A discussion of the influence of changes in operating conditions on the system head curve is provided in Appendix B. Guidelines for pump field tests are provided in Enclosure 2 of Appendix A. A summary of the results of field tests for the two major pumping stations of the DC and vicinity system is provided in Appendixes C and D.

10. The second component of the methodology concerns the determination of optimal pump combinations for specified operating conditions. Guidelines for determining the optimal combinations are provided in Enclosure 3 of Appendix A. A computer program developed for use in determining the optimal combinations is described in Appendix E.

11. The third component concerns the determination of an optimal operating policy for each pumping station for a given set of operating conditions. This is accomplished using a sophisticated computer program based on dynamic programming. A detailed description of the program is given in Appendix F. A summary of the theoretical basis for the program is provided in Enclosure 4 of Appendix A.

Overview

12. To investigate the feasibility of the proposed optimal pump operation methodology for a large, real system, the methodology was applied to the DC and vicinity water distribution system. Part II of this report presents a summary of the general characteristics of the DC system. Part III presents a

summary of the results of the pump field test for both the Dalecarlia and Bryant Street pumping stations. Part IV presents a summary of the proposed optimal pump methodology. Part V presents the results of the applications of the methodology to the second and third high-pressure zones of the DC system. Study recommendations and conclusions are presented in Part VI.

PART III: DESCRIPTION OF THE WASHINGTON, DC, [illegible]

[illegible]. The Washington Aqueduct Division [illegible] of the [illegible] Army [illegible] neer [illegible] District, Baltimore, has the responsibility [illegible] water to [illegible]. This is accomplished [illegible] points Dalecarlia and [illegible] the Potomac River. [illegible] water [illegible] [illegible]

[illegible]. Distribution of [illegible] water [illegible] the primary responsibility of the Water [illegible] the [illegible] the Department of [illegible] finished water is the responsibility [illegible] pumping stations of the system are the [illegible] the [illegible] Street pumping station [illegible]. Although [illegible] pumping stations exist, these two stations supply [illegible] flows [illegible] represent the major electric users of the system.

General Network Characteristics

15. Ground elevations in DC vary from under [illegible] ft* to [illegible] ft above mean sea level. To provide an average water pressure of about 50 psi over this range in elevation, the city is divided into seven pressure zones, each comprising a certain range of ground elevations. The pressure in each of these service areas is controlled to maintain a range of 40 to 80 psi depending upon whether a given location is near the upper or lower boundary of the service area. A map of the different service areas (pressure zones) is provided in Figure 1. A profile of the distribution system is shown in Figure 2.

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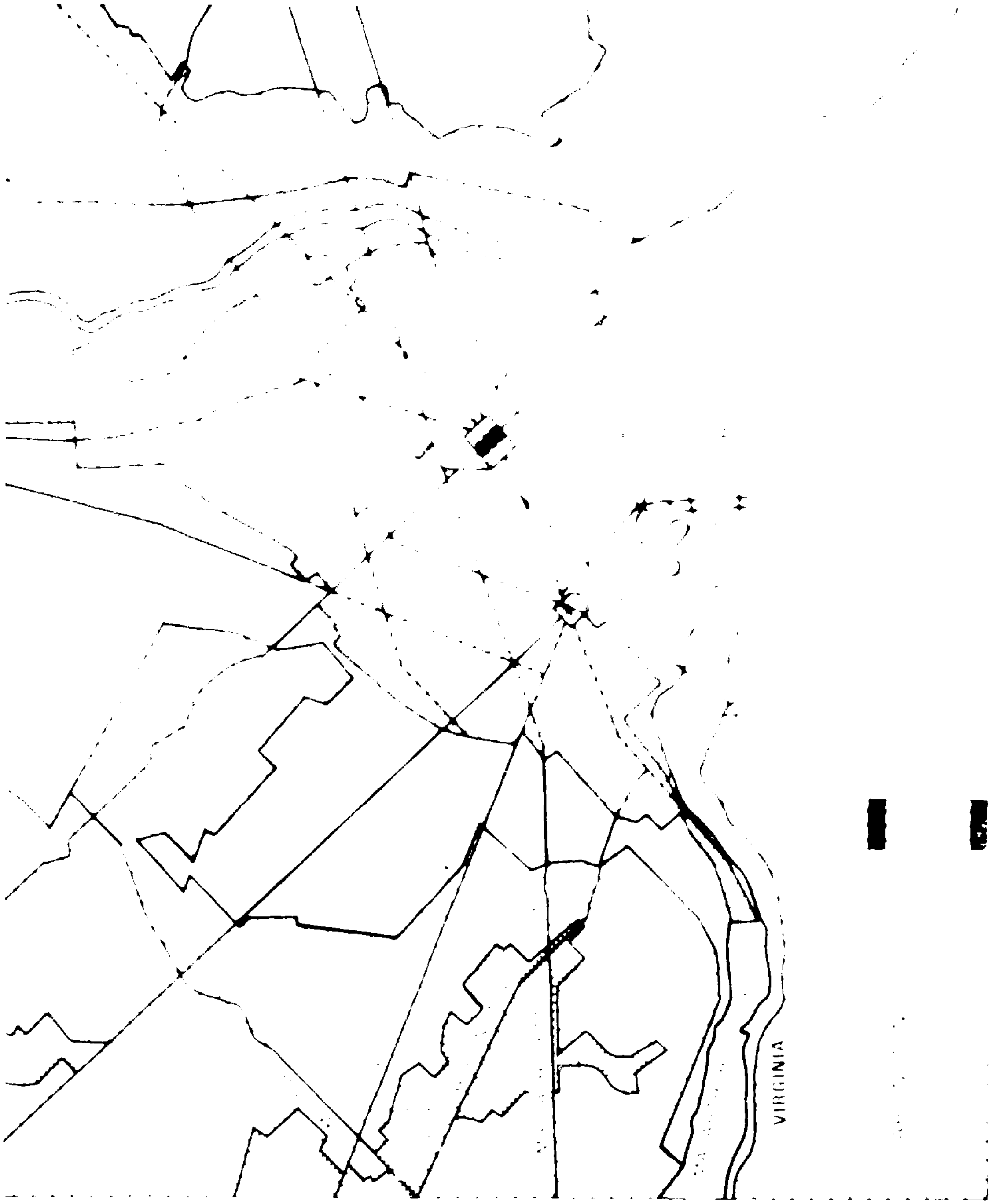
* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 4.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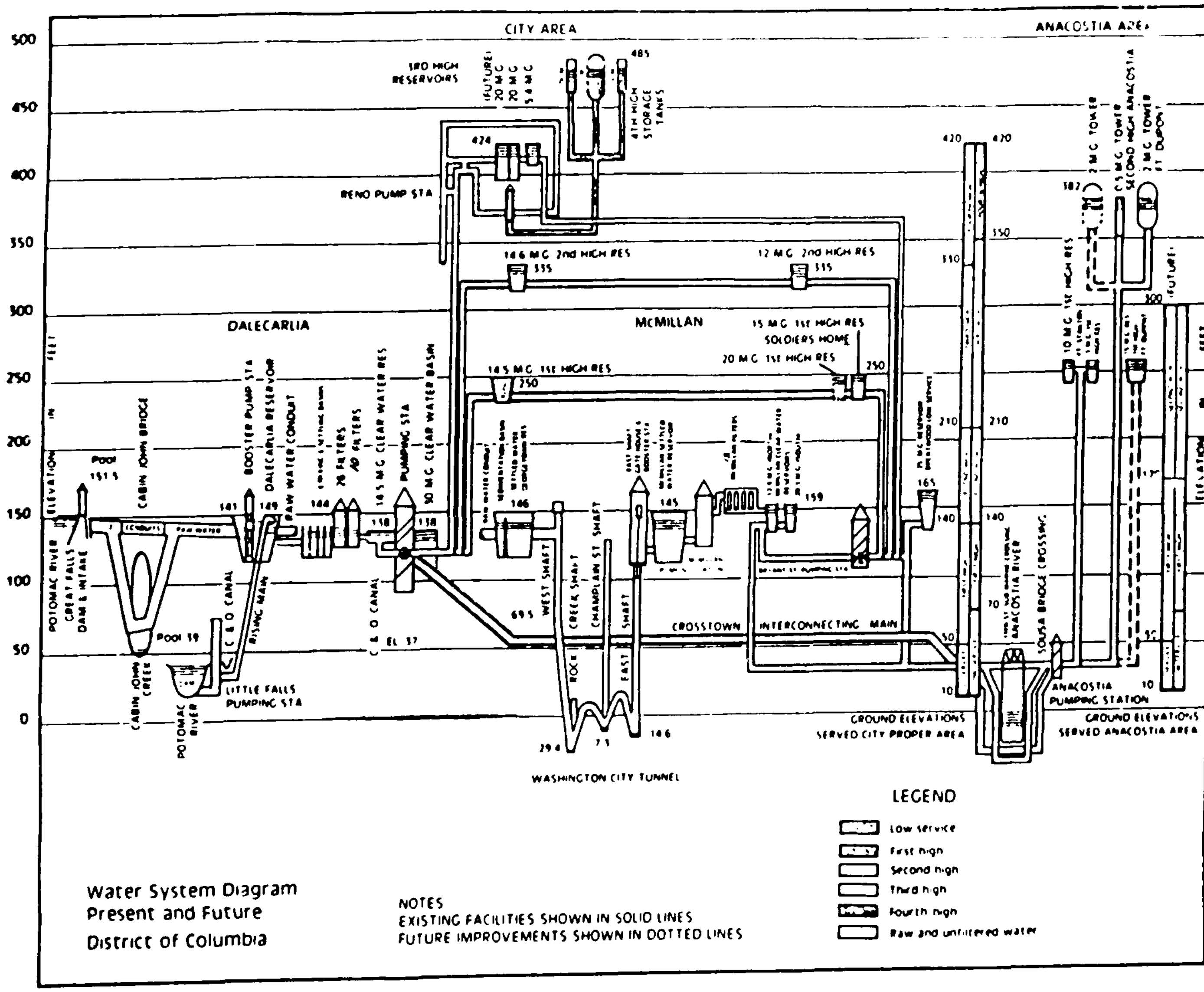


Figure 2. Profile of the District of Columbia distribution system

16. The low-service area, which includes the Federal District, is normally supplied by gravity from the McMillan treatment plant. Pressure is augmented by booster pumps from the Dalecarlia pumping station when required by demand. The Bryant Street pumping station is also capable of pumping to the low-service area. Low-service water also supplies lower elevations east of the Anacostia River and provides pump suction for the Anacostia booster pumping station. From this station, water is lifted to the Anacostia first and second high-service areas, not to be confused with the first and second high-service areas to the west of the Anacostia River.

17. The first, second, and third high-service areas, [illegible] progressively increasing ground elevations west of the Anacostia River, are supplied from both plants through the Dalecarlia and Bryant Street pumping stations and high-service reservoirs. The fourth high-service area, serving the highest elevations in the northwest section of DC, is supplied by pumping from the third high-service area into two elevated fourth area tanks, all at Fort Reno.

18. In addition to serving DC, the system supplies water to several cities in Virginia. Arlington is supplied from the third high-service area, and Falls Church is supplied from the second high-service area at the Dalecarlia pumping station. Federal buildings and reservations in Virginia, such as the Pentagon and Washington National Airport, are supplied through the FOWM system, which is tied into the first high-pressure zone at Key Bridge. A schematic of the system is provided as Figure 3.

19. With respect to the total system, the Dalecarlia and Bryant Street pumping stations operate in parallel. The additional booster pumps in the system act in series with respect to the Bryant Street and Dalecarlia stations. Because of this, and due to the magnitude of flows pumped by these two stations, only the Dalecarlia and Bryant Street pumping stations were used in applying the proposed optimal operation methodology. As a result, only the main system west of the Anacostia River and north of the Potomac was considered. The pump operation study was thus limited to the low-service and first, second, and third high-service areas. The fourth high-service area was included as a water user in the third high-service area, while the portion of DC east of the Anacostia River was considered to be a user in the low-service area. In addition, Arlington was included as a demand to the third high-service area, Falls Church was included as a demand to the second

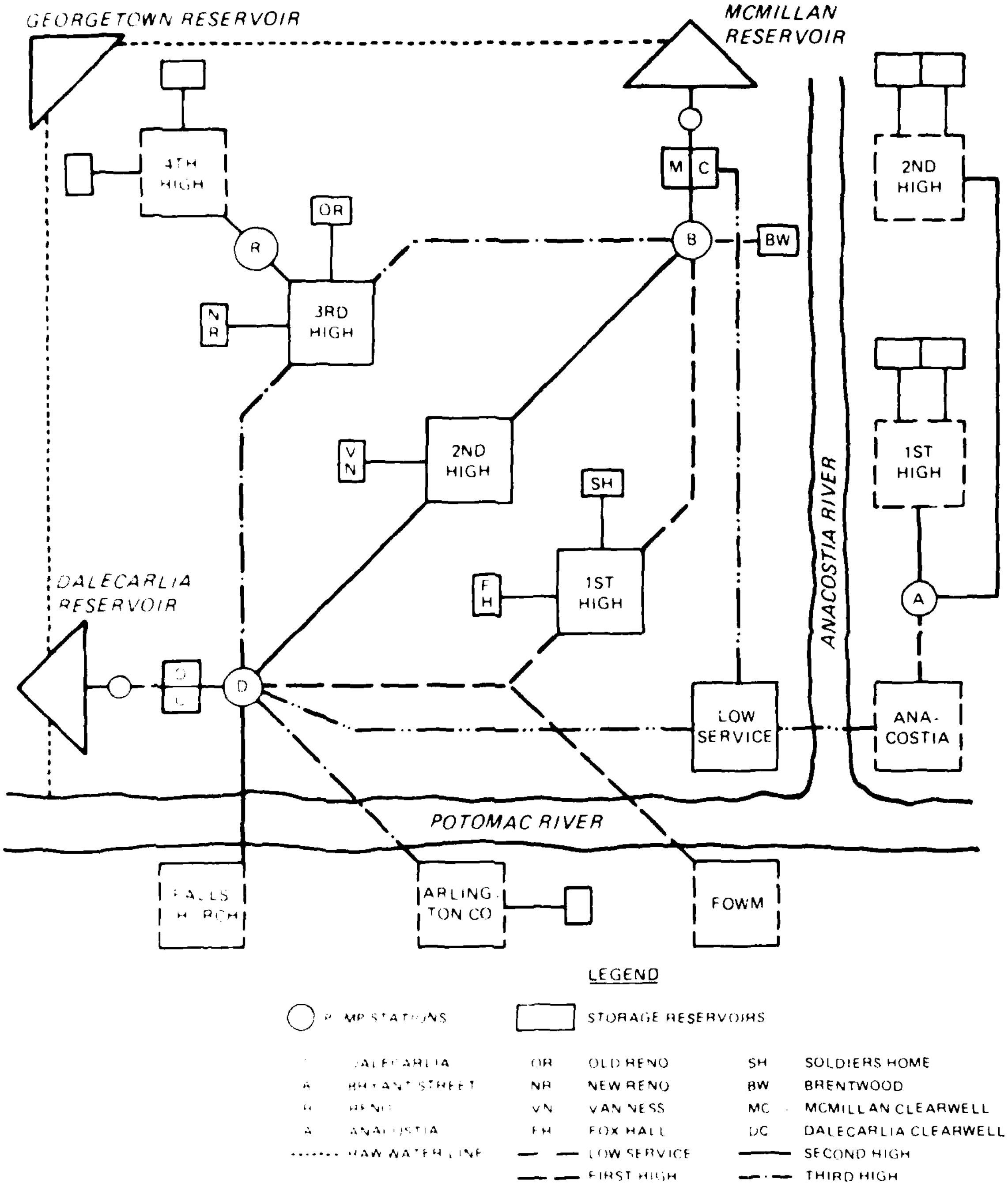


Figure 3. Schematic of DC system

high-service area, and the FOWM was included as a demand to the first high-service area.

Pipe Distribution Facilities

20. The distribution system consists of about 1,286 miles of water mains ranging from 2 to 78 in. in diameter. In addition, the Federal Government maintains and operates 12.3 miles of transmission mains within DC, which deliver water to the first, second, and third high-service area reservoirs. The water distribution system has approximately 18,500 valves, 8,800 fire hydrants, and 132,000 services.

Distribution Storage Facilities

21. Storage facilities in the modeled system (low-service and first, second, and third high-service areas) are either ground-level reservoirs or elevated steel tanks that are connected directly to the distribution system. Storage is designed to equalize pressure during periods of heavy consumption and to equalize pumping power loads. All distribution system reservoirs are constructed of reinforced concrete and are covered entirely. Including the clearwell storage, the modeled system has a total available storage of 127.7 million gallons. Elevations and capacities of the distribution storage facilities considered in this study are indicated in Table 1. Capacities of the storage facilities in the adjacent areas are indicated in Table 2.

Pumping Station Characteristics

22. The DC water distribution system contains four treated water pumping stations. The Dalecarlia pumping station is located at the Dalecarlia water treatment plant and is operated by the WAD. The total rated capacity of the Dalecarlia station is 477 million gallons per day (mgd). The Bryant Street, Anacostia, and Reno stations are located at various points in the system and are operated by the Department of Environmental Services. The total rated capacities of these three stations are 310, 157, and 22 mgd, respectively. Because the rated capacities of Anacostia and Reno are much

Table 1

Reservoir Characteristics

Service Area	Jurisdiction	Location	Capacity*	Max. Elev.	Min. Elev.
Low	DC WRMA	Brentwood Park	25.0	172	135
	WAD	North McMillan	12.9	159	135
	(clearwells)	South McMillan	20.3	159	135
		Subtotal	33.2		
First high	WAD	Foxhall Rd. NW	14.5	250	233
	DC WRMA	Soldiers Home	15.0	250	233
		Subtotal	29.5		
Second high	WAD	44th and Van Ness	14.6	335	318
Third high	WAD	New Reno NW	20.0	424	406
	DC WRMA	Old Reno NW	5.4	424	406
		Subtotal	25.4		
Total			127.7		

* In millions of gallons.

Table 2

Supplemental Reservoir Storage

Service Area	Capacity*
Fourth high	0.24
Anacostia first high	13.0
Anacostia second high	2.5
Falls Church	14.0
Arlington	32.0
FOWM	0.0
Total	61.74

* In millions of gallons.

smaller than Dalecarlia and Bryant Street, only Dalecarlia and Bryant Street pumping stations were analyzed in this study.

Dalecarlia pumping station

23. The present Dalecarlia pumping station, which was completed in 1958, consists of an underground reinforced concrete substructure, with a headhouse and electric substations aboveground. The underground portion is 205 ft long, 101 ft wide, and 47 ft deep, or approximately the depth of a four-story building. The station contains 15 Worthington, Inc., vertical-shaft centrifugal pumps, each of which is connected to a 4,000-V, 60-cycle, three-phase, water-cooled synchronous electrical motor. The pumps have a combined capacity of 477 mgd. All pumping units are designed to run safely in reverse rotation, in case of power failure, at maximum runaway speed, for 5 min under heads equal to the rated heads. Each pump is equipped with a cone valve for surge protection during start-up and shutdown. In addition, a butterfly valve that is located downstream may be used to throttle the pumps. Three 8- by 8-ft finished-water suction conduits supply the pumps. These conduits draw water from the 14.5- and 30-million gallon clearwater basins located immediately north of the pumping station.

24. The station is completely air-conditioned to provide dehumidification and temperature control. The control center includes a supervisory switchboard for operation control of all major pumping units and auxiliary equipment contained in the station. It also provides for remote control of a 325-mgd raw-water booster pumping station including all major pumping units and auxiliary equipment installed in the raw-water supply intake works and pumping station at Little Falls. Water from the Dalecarlia pumping station is supplied to the low-service area and the first three high-service areas. All 15 pumps in the Dalecarlia station were manufactured by Worthington, Inc., and are driven by synchronous motors. Pump service areas and capacities are listed in Table 3.

Bryant Street pumping station

25. The Bryant Street pumping station is located on the north side of Bryant Street between 2nd and 4th Streets, NW, and south of the McMillan reservoir. The station is operated to maintain predetermined minimum pressures on its pumped services. Suction is from 78-, 60-, and 48-in. connections from the McMillan clearwater basins. The maximum elevation of the McMillan clearwell is 159 ft.

Table 3

Dalecarlia Pumping Station Data

Pump	Serial No.	Model	Service Area	mgd	rpm	Head ft	Pump hp
1	1461376	30-MC-01-VRT	Low	50	514	50	500
2	1461375	30-MC-01-VRT	Low	50	514	50	500
3	1461374	30-MC-01-VRT	Low	50	514	50	500
4	1461379	26-NA-43-VRT	First high	35	600	145	1,000
5	1461378	26-NA-43-VRT	First high	35	600	145	1,000
6	1461377	26-NA-43-VRT	First high	35	600	145	1,000
7	1461382	18-NA-33-VRT	Second high	20	900	220	866
8	1461381	18-NA-33-VRT	Second high	20	900	220	866
9	1461380	18-NA-33-VRT	Second high	20	900	220	866
10	1461385	24-NA-38-VRT	Third high	27	900	300	1,590
11	1461384	24-NA-38-VRT	Third high	27	900	300	1,590
12	1461383	24-NA-38-VRT	Third high	27	900	300	1,590
13	1507034	24-NA-38-VRT	Third high	27	900	300	1,590
14	1507035	24-NA-38-VRT	Third high	27	900	300	1,590
15	1461386	24-NA-38-VRT	Third high	27	900	300	1,590

26. The Bryant Street Station was completely rehabilitated as of 1954. Pumps are protected from reversal on power failure by automatic closure of cone valves. Unlike the valves at Dalecarlia, these valves cannot be set at intermediate settings and must either be open or closed. The control panels include switchboards, flow and pressure recorders on pump discharge lines and nearby low-service trunk mains, and a piping diagram with position-indicating signals and controls for remote operation of important valves. Pump services and capacities for the Bryant Street pumping station are given in Table 4. Pumps 1-10 were manufactured by Worthington, Inc., while Pumps 11 and 12 were manufactured by Allis Chalmers. Pumps 1-10 operate in parallel; Pumps 11 and 12 operate in series.

Water Demand Schedule

27. The average water consumption by service areas of DC for fiscal year 1985 is given in Table 5. The average system demand for each pressure zone tends to vary depending upon the season and the day of the week. To

Table 4

Bryant Street Pumping Station Data

Pump	Serial No.	Model	Service Area	mgd	rpm	Head ft	Pump hp
1	1346785	26-NA-43-VRT	First high	35	514	110	800
2	1346786	26-NA-43-VRT	First high	35	514	110	800
3	1346784	26-NA-43-VRT	First high	35	514	110	800
4	1346781	30-MC-01-VRT	Low	35	514	45	325
5	1346782	30-MC-01-VRT	Low	35	514	45	325
6	1346783	30-MC-01-VRT	Low	35	514	45	325
7	1346787	24-NA-38-VRT	Second high	25	720	210	1,100
8	1346788	24-NA-38-VRT	Second high	25	720	210	1,100
9	1346789	18-NA-37-VRT	Third high	15	900	310	1,000
10	1346790	18-NA-37-VRT	Third high	15	900	310	1,000
11	23564	Size 20 × 18	Third high	20	720	155	325
12	23565	Size 20 × 18	Third high	20	720	155	325

illustrate the impact of these factors on the daily average system demand, the demands for each service area for four different days in 1986 are shown in Table 6. The selected days were 20 March (winter, weekday), 29 March (winter, weekend), 8 June (summer, weekend), and 11 June (summer, weekday). For each service area, daily demand patterns may vary considerably from day to day. As a result, a single representative demand pattern could not be obtained.

Electric Rate Schedule

28. All pumping stations receive their power from the Potomac Electric Power Company. The general electric rate schedule is provided as Appendix G.

Table 5

Water Consumption, Fiscal Year 1985

Service Area	Average Demand mgd
Low (including Anacostia)	72.5
First high (including FOWM)	38.7
Second high (including Falls Church)	39.8
Third high (including Fourth High and Arlington)	64.6
Total	215.6
FOWM	3.2
Arlington	23.3
Falls Church	16.0
Total	42.5

Table 6

Typical Demands, 1986

Service Area	Date	Water Use mgd
Low	(20 March 1986)	100
	(29 March 1986)	73
	(8 June 1986)	118
	(11 June 1986)	116
First high	(20 March 1986)	36
	(29 March 1986)	33
	(8 June 1986)	41
	(11 June 1986)	53
Second high	(20 March 1986)	38
	(29 March 1986)	40
	(8 June 1986)	52
	(11 June 1986)	50
Third high	(20 March 1986)	51
	(29 March 1986)	54
	(8 June 1986)	65
	(11 June 1986)	69
Total	(20 March 1986)	225
	(29 March 1986)	200
	(8 June 1986)	276
	(11 June 1986)	288

PART III: PUMP FIELD TEST RESULTS

29. Before the general methodology was applied to the DC and vicinity distribution system, each pump within the Dalecarlia and Bryant Street pumping stations was first field tested. The guidelines for the field tests are provided in Enclosure 2 of Appendix A. The following paragraphs present the results of those tests.

Dalecarlia Pumping Station

30. The pumps at the Dalecarlia station were field tested on 30 and 31 September 1985. At the time of the tests, Pumps 13 and 14 were out of service and were not tested. In addition to tests of the individual pumps, measurements were made where several multiple-pump combinations were operating.

31. Where possible, each pump was tested individually. The test was begun by first running the pump with the discharge line closed in order to determine the shutoff head. The valve in the discharge line was then opened partially and pressure, flow, and power readings were obtained. This process was continued until the valve was completely opened. Before each reading, time was allowed for the flow and head to reach steady-state conditions. After the valve was completely opened, it was slowly closed in incremental steps, and another set of readings was obtained.

32. Pressure readings were obtained using a calibrated Bordon tube pressure gage that was connected to the discharge line of the pump. The suction head was calculated using the elevation of the center line of the pump and the clearwell elevation. For the Dalecarlia station, the clearwell is above the pump center line which, coupled with short, large suction lines, results in a positive suction head. The center-line elevation of the discharge lines of each pump is 106 ft. During the field tests the clearwell elevation varied between 134 and 135 ft.

33. Power and flow readings were obtained using the instrumentation in the control room. The flow meters were calibrated on 18 October 1985. All flow meters were found to be within acceptable limits, with the exception of the meter on the third high-service area, which was found to be 16.9 percent high at the 35-mgd rate. As a result, flow readings for the pumps on the

third high-service system were adjusted accordingly. The results of the field tests for each pump are provided in Appendix C.

34. After the field data were collected for each pump, the results were compared with the most recent pump curves available from the manufacturer. According to available records, these curves were developed in 1959. Comparisons of the percent differences between measured and manufacturer's values of flow rate and power as a function of pump head are given for each pump in Figure 4. As shown, the percent differences for the power readings are generally much smaller than the flow rate readings. The reason for the much larger deviation in the pump head versus flow rate curves can be explained in terms of changes in the pump characteristics over time or in terms of measurement errors. Since the pump head versus power curves matched very well, it was concluded that the deviations were due to measurement error.

35. To determine the possible cause of this apparent error, a careful review of the data was performed. Based on this examination it was concluded that the measured flow rates were the most likely source of the error. This conclusion was based on the accuracy of the measuring equipment. While the pressure and horsepower measurements were obtained directly from power and pressure gages, the flow rate readings were obtained from circular pen charts. The range of measurement error was thus much greater for the flow rate measurements than for the power and pressure measurements.

36. Since the measured flow rates were concluded to be in error, wire-to-water efficiencies based on the measured data could not be determined. However, since the measured pump head versus power curves showed such a good correlation with the manufacturer's pump head versus power curves, and since this relationship is a function of flow rate, it was concluded that the actual wire-to-water efficiencies were most likely very close to the original manufacturer's values. As a result, all wire-to-water efficiencies were obtained using the measured values of pump head and the original manufacturer's efficiency curves.

37. In addition to testing the efficiency of each pump under conditions of single-pump operation, the efficiency of each pump while operating in combination with other pumps was also determined. The maximum observed efficiency for each pump for different pump combinations is illustrated in Table 7.

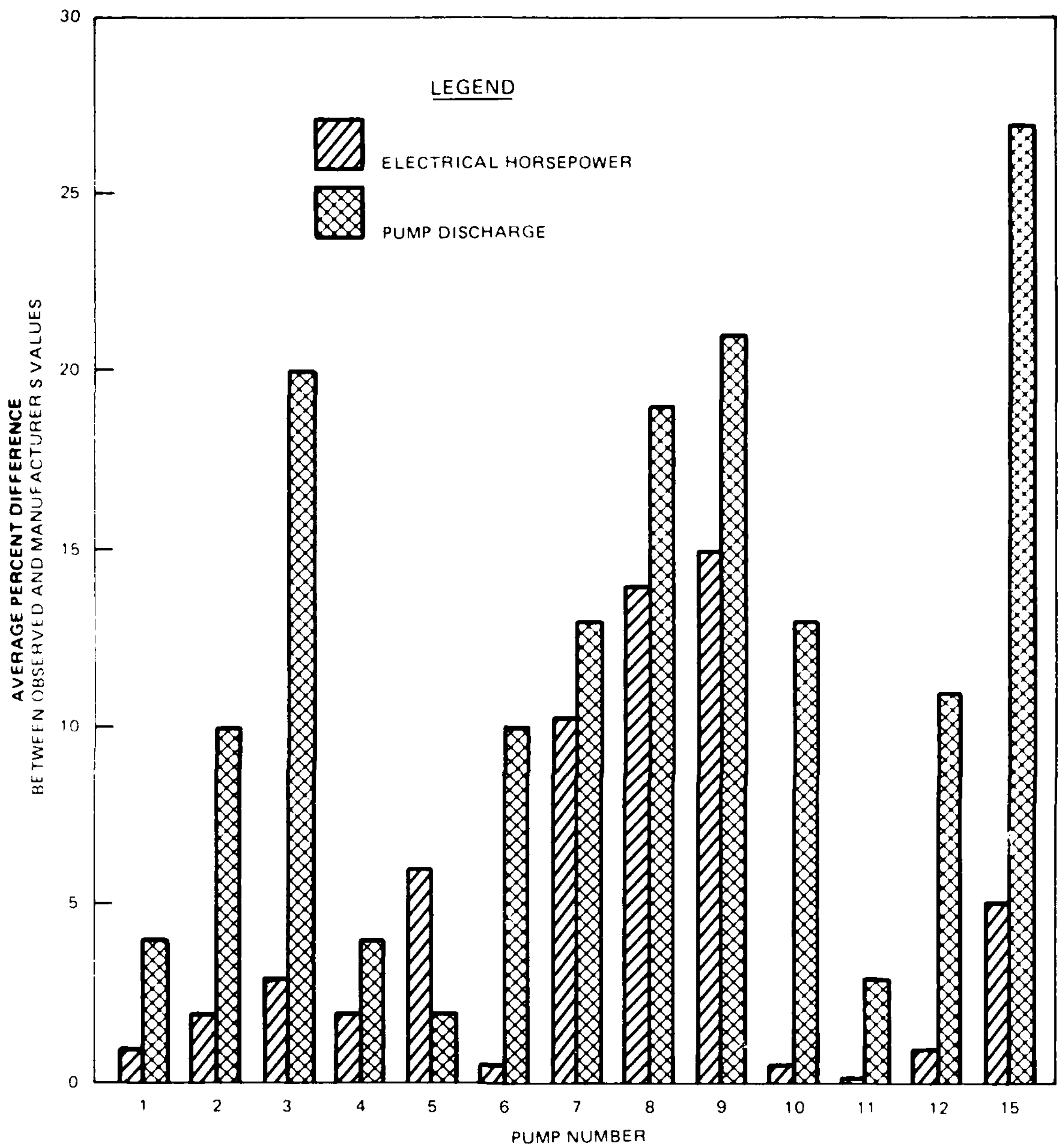


Figure 4. Summary of field test results for Dalecarlia pumping station

Bryant Street Pumping Station

38. The pumps at the Bryant Street station were field tested on 31 September 1985. At the time of the test, results were not obtained for Pump 1. In addition to single pump tests, tests were also performed for several combinations of multiple-pump operations.

Table 7

Observed Wire-to-Water Efficiencies for Multiple-Pump Combinations, Dalecarlia Pumping Station

Pump No.	Number of Pumps Operating		
	1	2	3
1	0.84	0.75	-
2	0.82	0.74	-
3	0.84	-	-
4	0.84	0.85	-
5	0.87	-	-
6	0.85	0.85	-
7	0.70	0.84	-
8	0.82	0.85	-
9	0.82	0.85	-
10	0.85	0.86	0.85
11	0.85	0.86	0.84
12	0.85	0.85	0.85
13	-	-	-
14	-	-	-
15	0.85	0.84	0.85

39. The pumps at Bryant Street station were not equipped with valves on the discharge lines that could be throttled in incremental steps. Instead, the pump was started up with the cone valve closed. Pressure, flow, and power readings were then obtained as the valve was opened. Given the resulting transient flow situation, only the readings with the pump closed and the pump fully opened can be accredited with any reliability.

40. Pressure readings at the Bryant Street station were obtained in the same manner as at the Dalecarlia station. As before, the suction head was obtained using the clearwell elevations and center line of the pumps. For the Bryant Street station, the center-line elevation of the pumps was 118 ft. During the tests the clearwell elevation varied between 155 and 157 ft. Transient flow readings were obtained using pressure transducer instrumentation.

Steady-state readings from the pressure transducer were augmented with the readings from the control room instrumentation.

41. Unlike the Dalecarlia station, the Bryant Street station pumps are driven with squirrel cage induction motors. As a result, the instrumentation in the control room did not provide power readings in kilowatts directly. Instead, instrumentation was provided for power in kvars and current in amps. However, using this information and the voltage drop across the pumps, the resulting kilowatt power can be determined (see Appendix A). For the Bryant Street station, the voltage drop across the pumps is equal to 2.3 kV. The pump field test results for the Bryant Street station are given in Appendix D.

42. As before, after the field tests were completed, the results were compared with the data obtained from manufacturers' pump curves. Percent differences were obtained for different values of wire-to-water efficiency. For the Bryant Street station, only three manufacturers' pump curves could be obtained (i.e., for Pumps 6, 7, and 8). As a result, wire-to-water efficiency comparisons could be obtained for only these pumps.

43. As with the Dalecarlia station, several multiple-pump combination tests were also conducted. The results of these tests are shown in Table 8.

Table 8

Observed Wire-to-Water Efficiencies for Multiple-Pump Combinations, Bryant Street Pumping Station

Pump No.	Number of Pumps Operating: 1	2	3
1	-	0.76	-
2	0.76	0.66	-
3	0.73	0.77	-
4	0.33	0.54	-
5	0.84	0.52	-
6	0.88	-	-
7	0.80	-	-
8	0.56	-	-
9	0.76	-	-
10	0.85	-	-
11	0.67	-	-

Summary

44. In general, all the pumps in the Dalecarlia station appear to be operating near their peak efficiencies. These results are somewhat clouded, however, due to accuracy problems encountered in measuring flow rate. Since the pump head and power readings are accurate, the pump efficiency readings obtained are at least consistent. In general, increasing the number of pumps operating in parallel does not seriously affect the peak pump efficiencies of the individual pumps. This indicates that the pumps were correctly selected.

45. The field test results for the Bryant Street pumping station are probably less accurate than the results obtained for the Dalecarlia station. This loss of accuracy resulted in part from the inability to obtain multiple readings by throttling the pumps. In addition, four parameters had to be measured for each pump at Bryant Street while only three parameters had to be measured for the Dalecarlia pumps. Although manufacturers' pump curves were obtained for all the Dalecarlia pumps, curves were available for only three pumps in the Bryant Street station. As a result, an evaluation of the effect of pump wear on the Bryant Street pumps was greatly limited.

46. In general, the majority of the Bryant Street pumps appeared to be operating at reasonably efficient levels (although lower than Dalecarlia). Two exceptions to this trend were Pumps 4 and 8, which were operating at much lower efficiency levels. It should be noted, however, that only two measurements were obtained for these pumps and, in both cases, these readings were at extremes in their operating range.

47. This part presents a summary of a methodology for determining the optimal pump operation policy for a given distribution system. The operation policy for a pump station is a set of rules or guidelines that indicates when a particular pump or group of pumps should be turned on and off over a specified period of time. The optimal pump operation policy is defined as that schedule of pump operations that will result in the lowest total operating cost for a given set of operating conditions.

48. For systems whose hydraulics are dominated by elevated storage, the pump operation will be directly related to the water level (or storage) in the tank. As a result, any optimal pump operating policy will have with it an associated optimal tank trajectory. The optimal tank trajectory is a curve that indicates the optimal tank level at a given time during a specified operating period.

49. The optimal pump operation problem can be subdivided into two subproblems. The first problem involves determining the optimal pump combination required to produce a specified tank transition. This problem will be defined the "optimal pump combination problem." The second problem involves determining the optimal tank trajectory over a specified period of time for a given set of operating conditions. This problem will be defined the "optimal tank operation problem." Together, both problems constitute the optimal pump operation problem. Solution methodologies for all three problems are discussed in the following sections.

Optimal Pump Combination Problem

50. The optimal pump combination problem may be solved using a two-step solution methodology. Each step requires the application of a different computer program, as shown in Figure 5. The first step produces a set of curves that are used in the second step. The final step produces the desired optimal pump combination. A discussion of each step is provided in the following sections.

Development of pump operation curves

51. The first step involves the development of a set of pump operation curves for each possible pump combination. These curves can be constructed

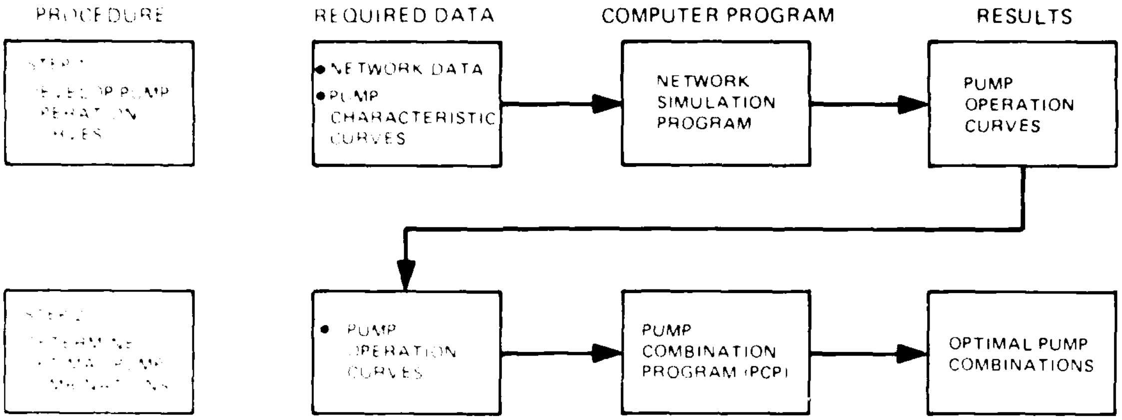


Figure 5. Optimal pump combination flowchart

using the results obtained from the application of a network analysis program to a model of the distribution system for a wide range of operating conditions. Guidelines for the construction of the curves are provided in Enclosure 3 of Appendix A. Two groups of operating curves are required. The first group of operation curves are called tank levels versus flow rate (TLF) curves (see Figure 6). Normally, three different curves are required for each pump combination. For a given pump combination and average tank level, these curves can be used to determine the flow supplied by each pump combination over a specified time interval. The second group of curves are called tank level versus unit cost (TLC) curves (see Figure 7). As with the TLF curves, three different curves are required for each different pump combination.

Determination of
optimal pump combinations

52. For a given pump combination and average tank level, the pump operation curves can be used to determine the operating cost associated with a particular pump combination. To determine the optimal pump operating policy for a given pumping station, many possible pump combinations must be examined for a wide range of operating conditions. The TLF and TLC curves provide a simplified way for approximating the hydraulics and operating costs of a particular pump combination without resorting to a complete hydraulic and economic analysis.

53. The optimal pump combination for a particular operating condition can be determined by applying an optimal pump combination program (PCP) developed especially for this purpose. A description of the program is given in

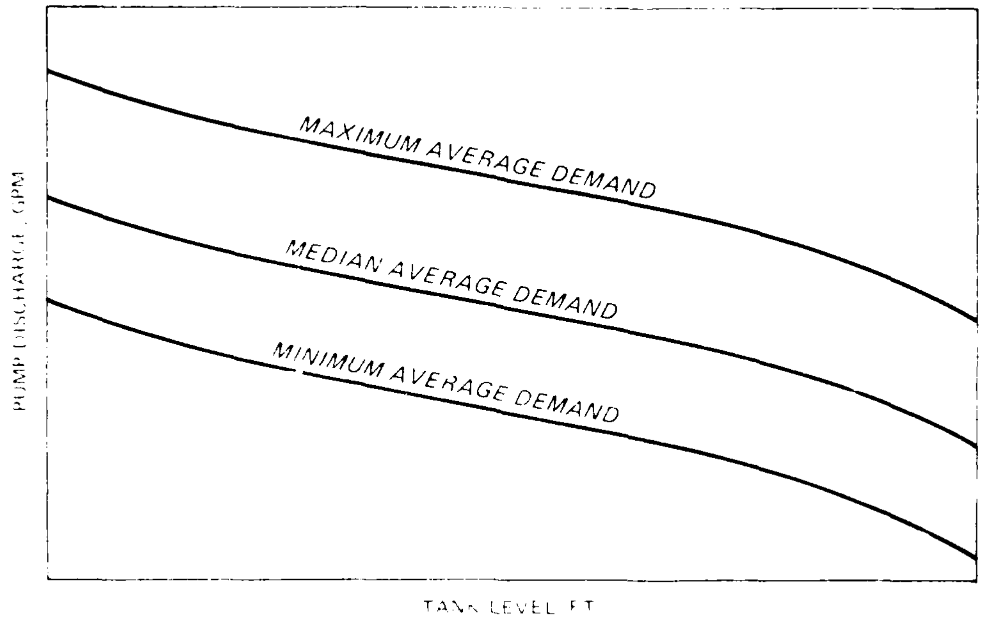


Figure 6. Pump operation curves - tank level versus flow rate

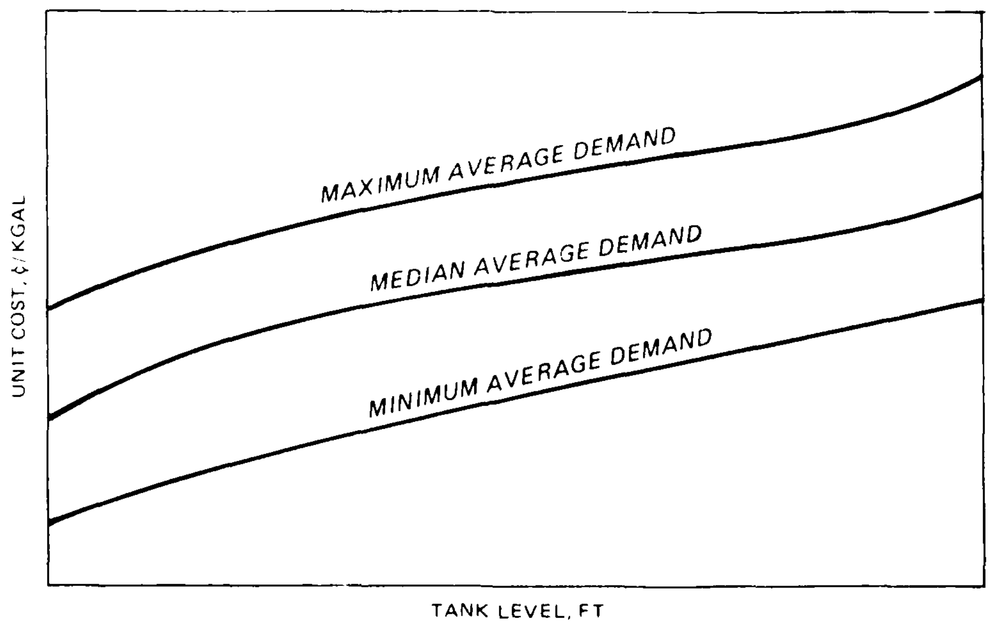


Figure 7. Pump operation curves - tank level versus unit costs

Appendix E. A discussion of the theory behind the program is given in Enclosure 3 of Appendix A. Basically, the program uses the TLF and TLC curves as input data and then enumerates all the possible pump combinations that will meet a specified set of operating conditions. Once the combinations have been identified, the program determines the cost associated with each combination and ranks the combinations from the least to most expensive.

Optimal Tank Operation Problem

54. The optimal tank operation problem may also be solved using a two-step solution methodology. As before, each step requires the application of a different computer program, as shown in Figure 8. Each step of the methodology is discussed in the following sections.

Development of cost operation curves

55. The first step involves the development of a set of cost operation curves (see Figure 9). These curves can be used to determine the minimum cost required to change from one tank level to another over a specified period of time for a required flow rate from the pumping station. The required flow rate will be equal to the sum of the system demand plus (or minus) the flow

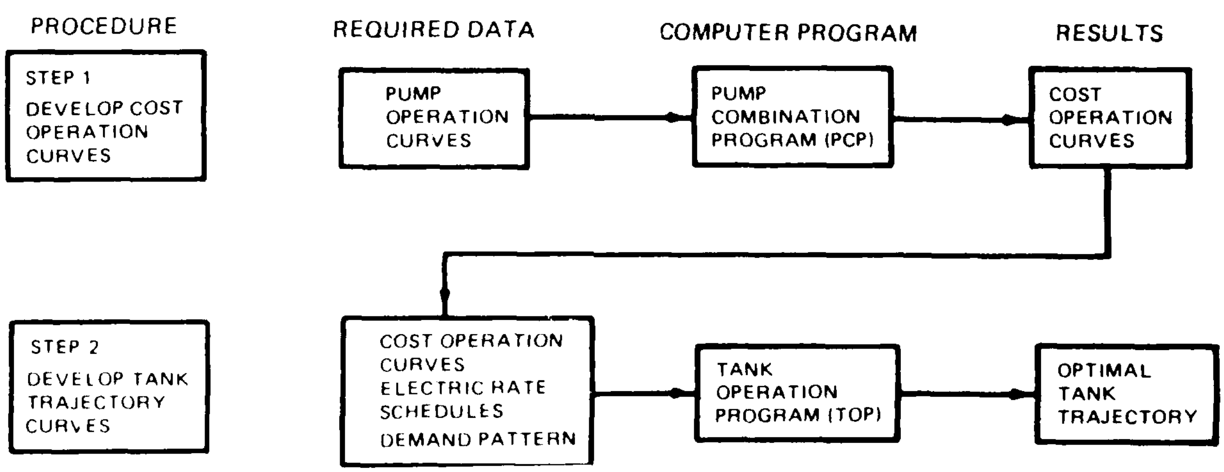


Figure 8. Optimal tank operation flowchart

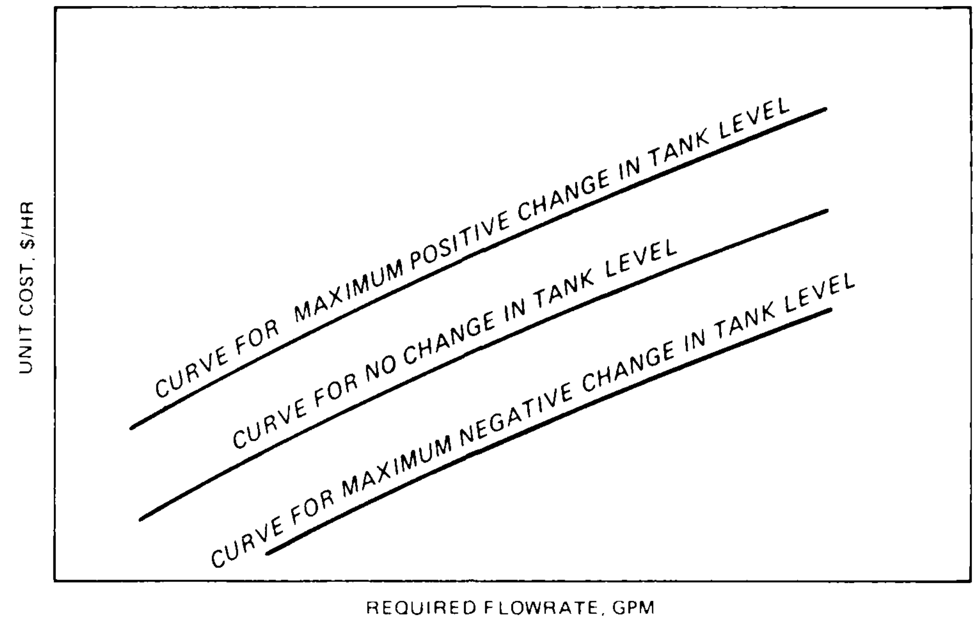


Figure 9. Cost operation curves

rate equivalent of the change in storage in the tank. These curves are used by an optimal tank trajectory program for determining the optimal tank trajectory for a given tank. Three curves are normally required to cover the range of possible operating conditions. Each curve may be obtained by fitting a polynomial through a series of data points. Normally a quadratic curve is sufficient to describe the variation of unit cost and flow rate for a given tank transition. As a result, only three points are needed to construct each curve. Each point on a particular curve represents the minimum cost required to supply a given flow rate from the pump station while operating over the specified tank transition. This minimum cost can be obtained by determining the optimal pump combination associated with the particular operating condition. The optimal pump combination can be determined by applying the PCP discussed previously.

Determination of the optimal tank trajectory

56. The second step involves the determination of the optimal tank trajectory. This is accomplished using an optimal tank operation program (TOP).

A description of the program is given in Appendix F. A discussion of the theory behind the program is given in Enclosure 4 of Appendix A. The TOP uses dynamic programming to determine the optimal tank trajectory for a given set of operating conditions. In applying dynamic programming to the optimal tank trajectory problem, the overall problem is broken into a series of subproblems. This is accomplished by dividing the operating period (typically a day) into smaller time units (typically hours). The dynamic program then solves a series of subproblems to determine the overall or global solution. Each subproblem involves determining the best possible tank transition for a given time segment (e.g., 1 hr). For each subproblem, the dynamic program must evaluate numerous potential tank transitions. The cost associated with each tank transition must be determined by solving the optimal pump combination problem discussed above. One way to handle this problem would be to embed the PCP directly into the TOP. Although this is a possible approach, it does not represent a computationally feasible alternative. Another approach would be to approximate the results of the PCP with a series of curves. This is accomplished through the construction of the cost operation curves discussed above. For any given tank transition associated with a particular subproblem, the required flow rate can be determined. As a result, the cost associated with the required flow rate and an associated tank transition can be determined directly from the cost operation curves.

Optimal Pump Operation Problem

57. The advantage of using the cost operation curves in TOP as an alternative to directly using PCP is that the resulting program is much more computationally efficient. The disadvantage is that TOP yields only the optimal tank trajectory; the pump operation policy needed to produce the trajectory is not determined. However, once the optimal tank trajectory has been determined, the pump operation policy required to produce the trajectory can be determined by reapplying PCP for each individual tank transition. Since the initial and final tank levels for a given period are now known (from TOP), PCP can be used to determine the optimal pump combination required to produce each such transition. As a result, the optimal pump operation problem can be solved by combining the solution methodologies for both the optimal pump

combination problem and the optimal tank operation problem into a single solution methodology. The resulting methodology is summarized in Figure 10.

Summary

58. The general optimal pump operating methodology can be summarized in four steps:

- a. Develop pump operation curves for each possible pump combination associated with a given service area using the procedures outlined in Enclosure 3 of Appendix A.
- b. Develop cost operation curves for each service area using the curves developed in step a and the pump combination program discussed in Appendix E.
- c. Determine the optimal tank trajectory for each service area for a specified demand pattern using the curves developed in step b and the tank operation program discussed in Appendix F.

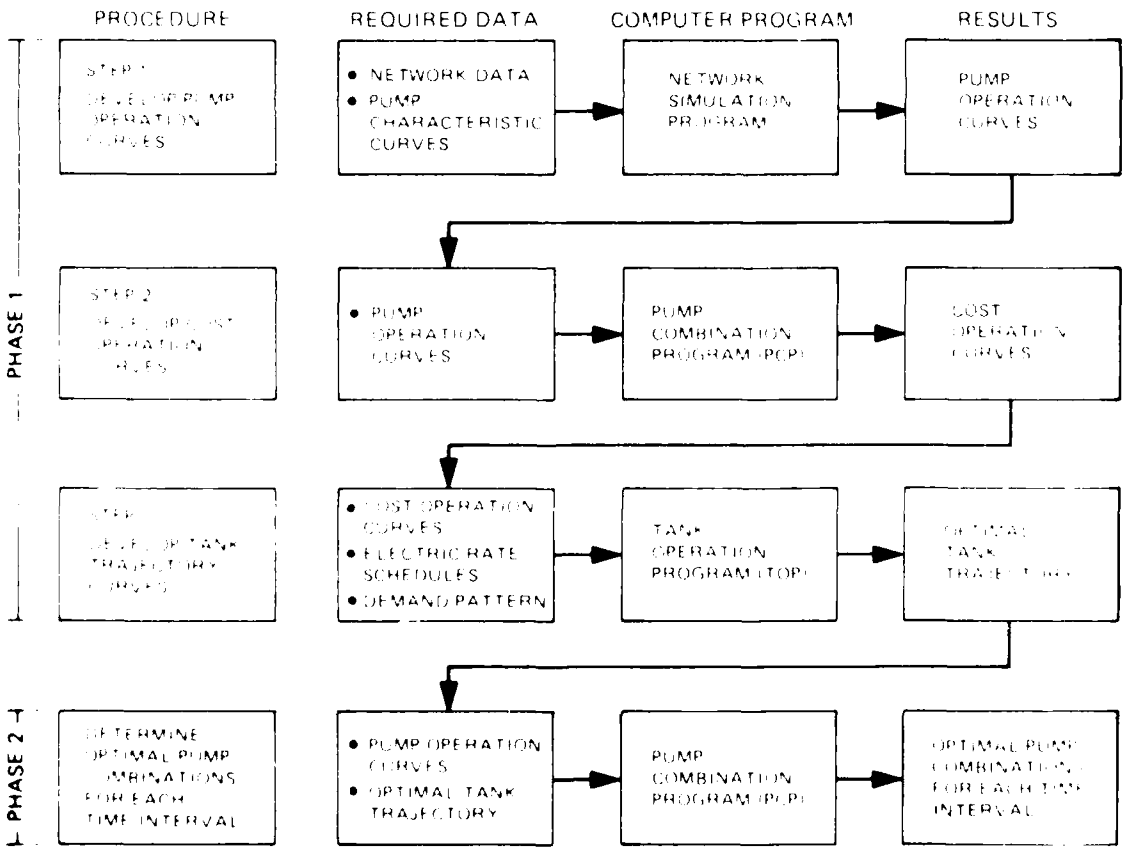


Figure 10. Optimal pump operation flowchart

d. Determine the optimal pump combinations for each time interval in a given day using the pump combination program (Appendix E), the pump operation curves developed in step a, and the optimal tank trajectory determined in step c.

PART V: APPLICATION OF METHODOLOGY TO WASHINGTON, DC, SYSTEM

59. To illustrate the applicability of the optimal pump operating methodology, the methodology was applied to two of the four pressure zones (service areas) of the primary DC distribution system. The zones selected for this study were the second and third high-pressure zones. Although the general methodology could have been applied to the low-service and first high-pressure zones as well, the second and third high-pressure zones were selected based on the fact that no gravity flows from the clearwells were present (as in the low-service zone) and only one tank policy was needed (two are needed for the first high-pressure system). Although both of these factors could have been incorporated into the methodology as discussed in a later section, their absence simplified the problem.

Second High-Pressure Zone

60. The second high-pressure zone supplies water to that portion of DC west of the Anacostia River, with ground elevations between 140 and 210 ft. In addition, Falls Church is also supplied from this zone. During 1985, the average daily demand for the second high-pressure system was 39.8 mgd. On the average, 16.0 mgd was supplied to Falls Church. A schematic of the system is shown in Figure 11.

61. The second high-pressure zone is supplied by three pumps in the Dalecarlia pumping station and two pumps in the Bryant Street station. (The physical characteristics of these pumps are described in Part II.) The pressure head in the system is provided by a single 14.6-million gallon storage tank located at 44th Street and Van Ness. The maximum elevation of the tank is 335 ft, while the minimum elevation is 318 ft. Under normal operating conditions the water level in the tank is maintained between 334 and 326 ft.

Third High-Pressure Zone

62. The third high-pressure zone supplies water to that portion of the District of Columbia west of the Anacostia River, with ground elevations between 210 and 350 ft. Arlington and the fourth high-pressure zone are also supplied from the third high-pressure system. During 1985 the average daily

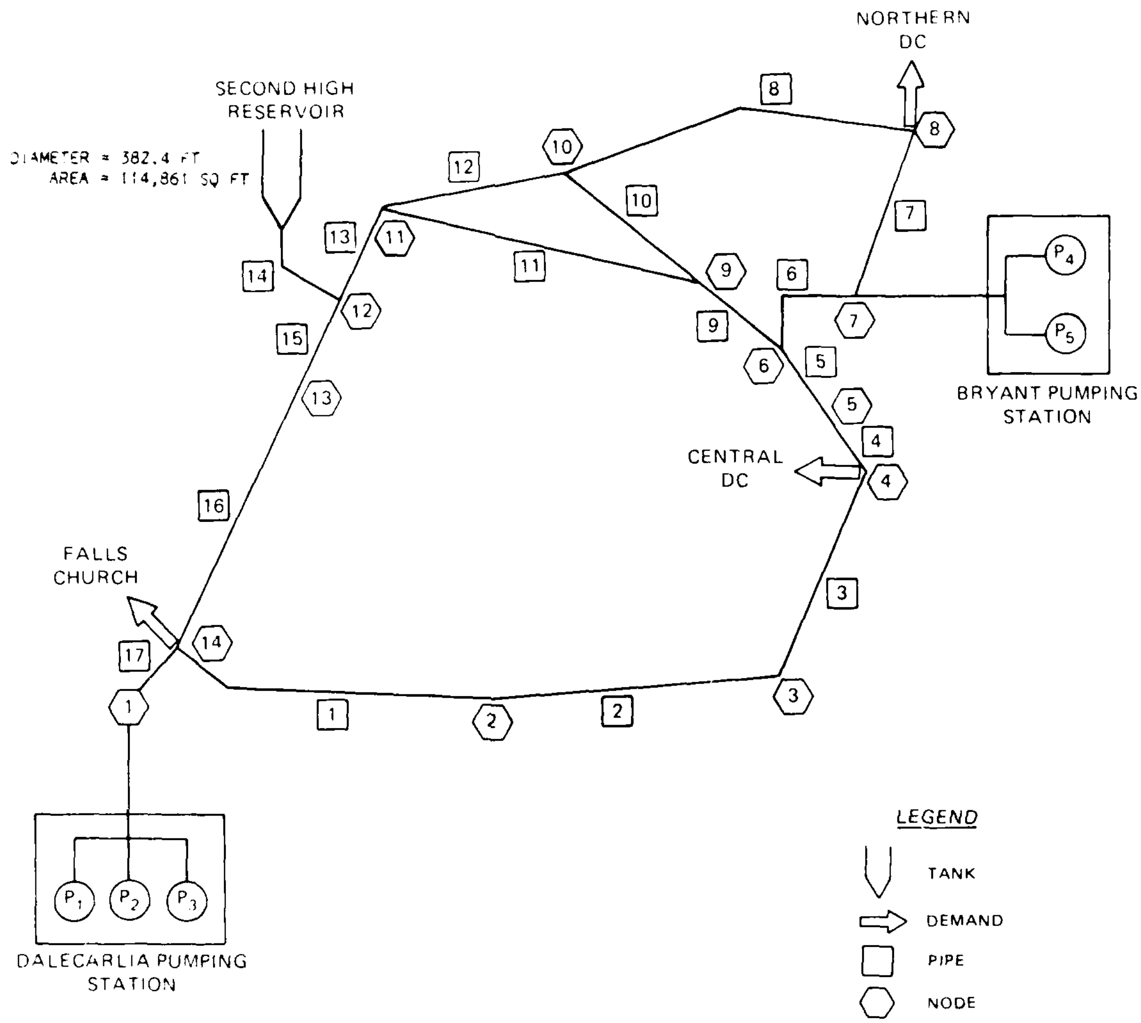


Figure 11. Schematic of second high-pressure zone

demand for the third high-pressure system was 64.6 mgd. Of this amount, 23.3 mgd was used by Arlington. A schematic of the system is shown in Figure 12.

63. The third high-pressure zone is supplied by six pumps in the Dalecarlia pumping station and three pumps in the Bryant Street station. (The physical characteristics of these pumps are described in Part II.) The pressure head in the system is provided by two tanks located at the Reno pumping station. The combined capacity of the tanks is 25.4 million gallons. The maximum elevation of the tanks is 424 ft, while the minimum elevation is 406 ft. Under normal operating conditions the water level in the tanks is maintained between 424 and 414 ft.

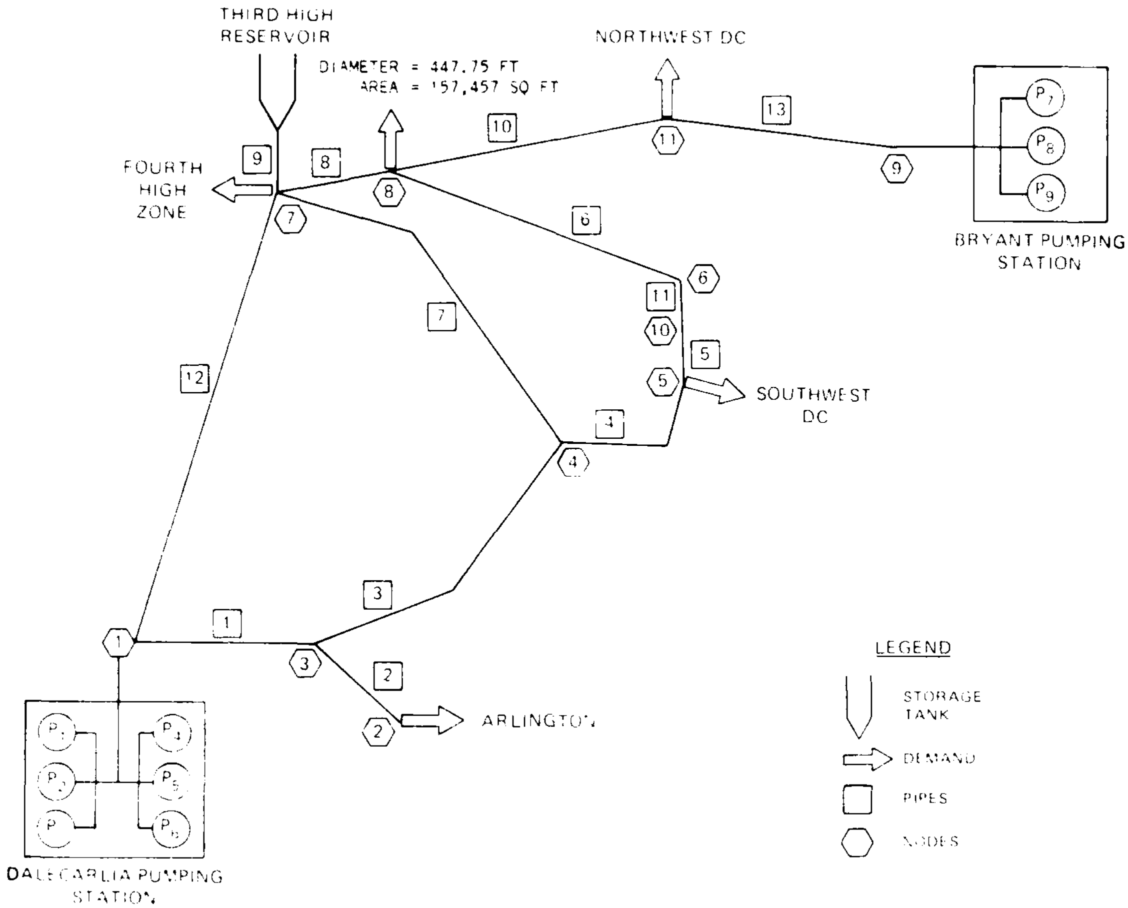


Figure 12. Schematic of third high-pressure zone

System Demand and Electric Rate Schedules

64. The optimal pump operation policy for a particular system will change from day to day depending on: (a) the electric rate schedule and (b) the system demand schedule. The electric rate schedule for both the Dalecarlia and Bryant Street pumping stations varies dependent upon the season (winter versus summer) and the particular day of the week (weekend versus weekday). The system demand schedule also varies considerably depending upon the season and day of the week.

65. In order to examine the impact of these factors on the optimal pump operation policy, the optimal pump operation methodology was applied to both the second and third high-pressure systems for four different days in 1986: 20 March (winter, weekday); 29 March (winter, weekend); 8 June (summer,

weekend); and 11 June (summer, weekday). The electric rate schedules for these 4 days for both pressure zones are shown in Figures 13-16. The electric rate schedules were constructed from the Potomac Electric Power Company time-metered general service schedule shown in Appendix G.

66. The system demand schedules for these 4 days for both pressure zones are shown in Figures 17-24. The flow rate demand for each hour was obtained by performing a mass balance of pump flows and tank flows using the following equation:

$$Q(\text{demand}) = Q(\text{pump}) \pm \frac{\Delta V(\text{tank})}{\Delta t}$$

where

Q(demand) = flow rate demand

Q(pump) = pump flow

ΔV(tank) = change in tank volume during time interval Δt

Values of Q(pump) and Δv(tank) were obtained from daily operation records for each system.

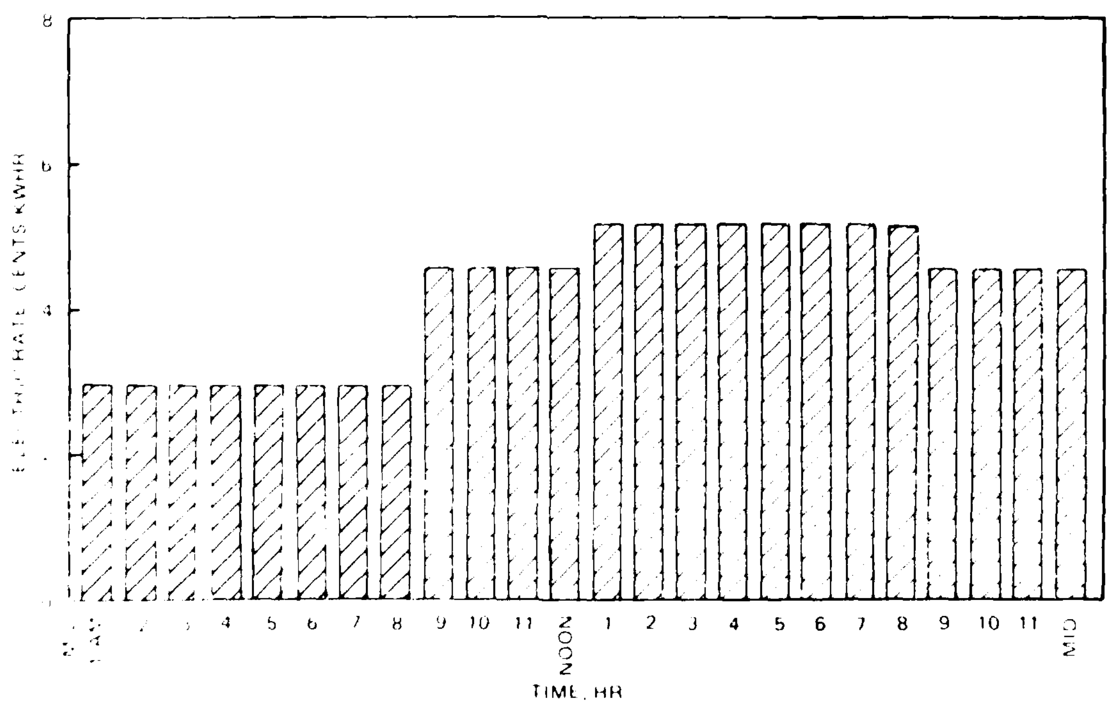


Figure 13. Electric rate schedule, Thursday, 20 March 1986

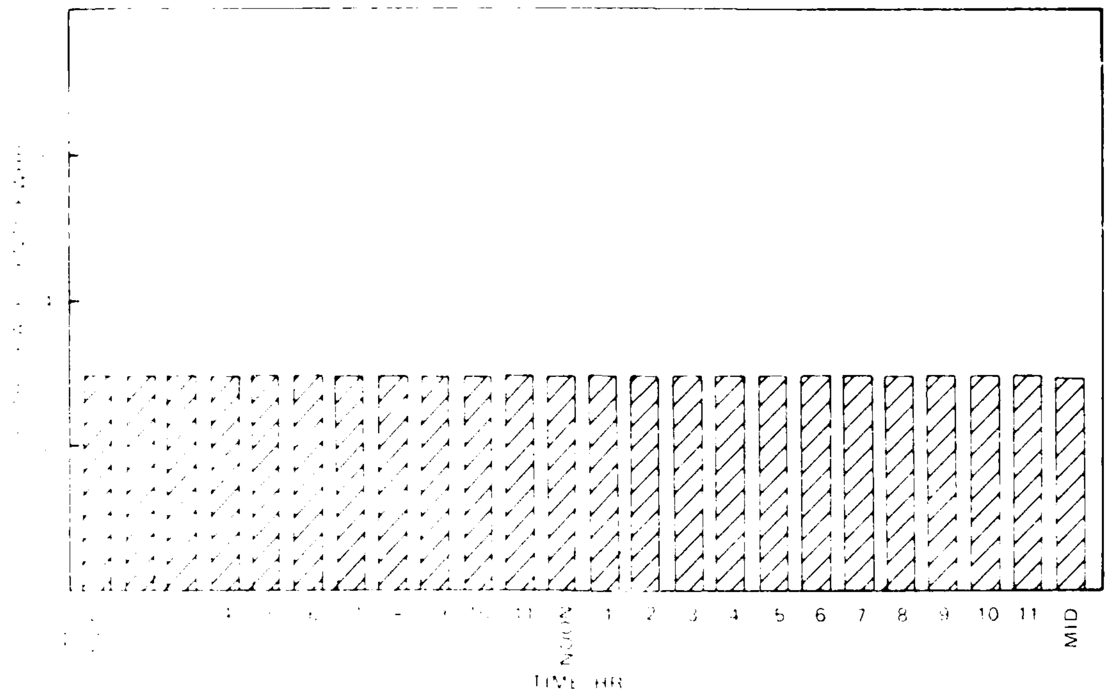


Figure 14. Electric rate schedule, Saturday, 29 March 1986

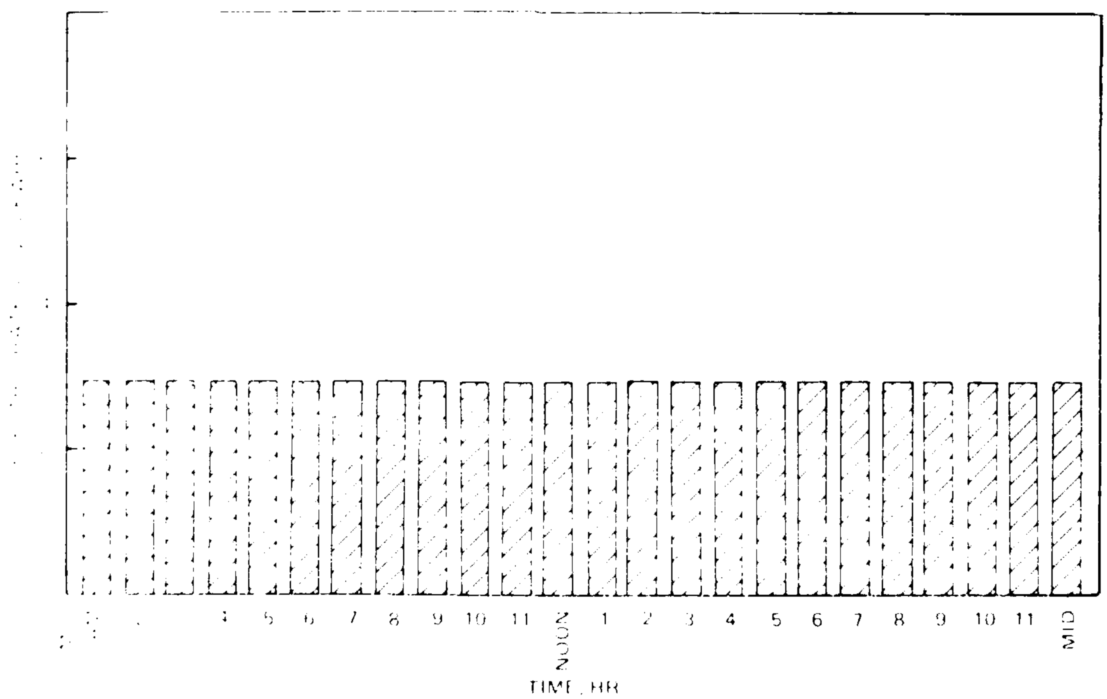


Figure 15. Electric rate schedule, Sunday, 8 June 1986

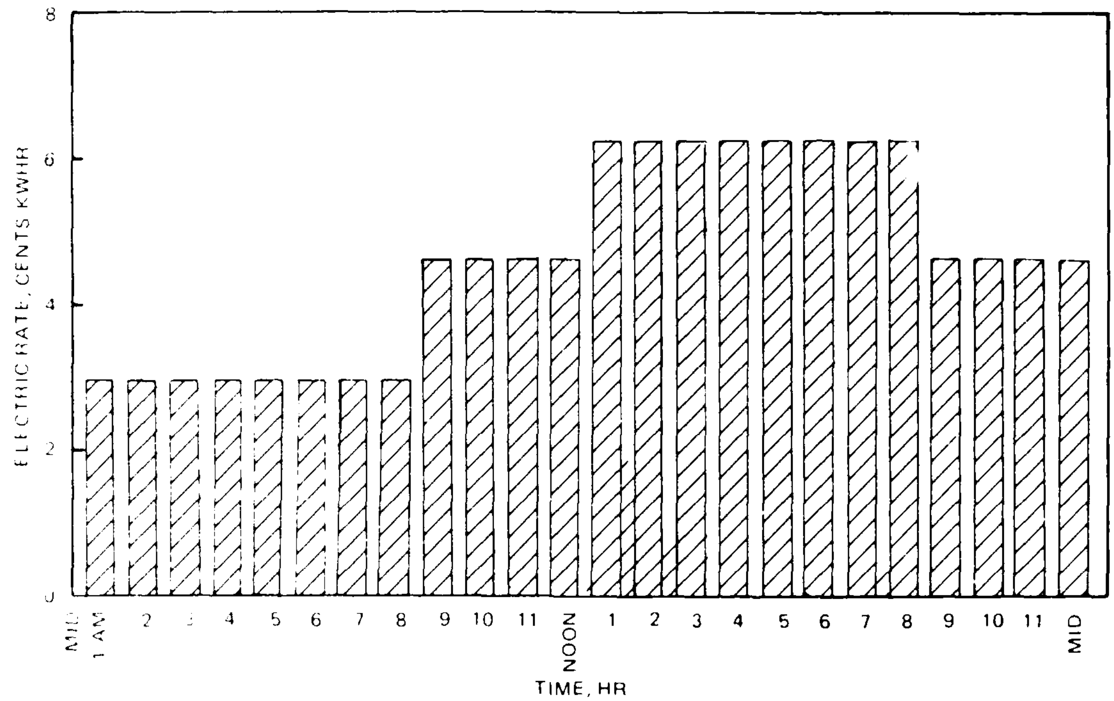


Figure 16. Electric rate schedule, Wednesday, 11 June 1986

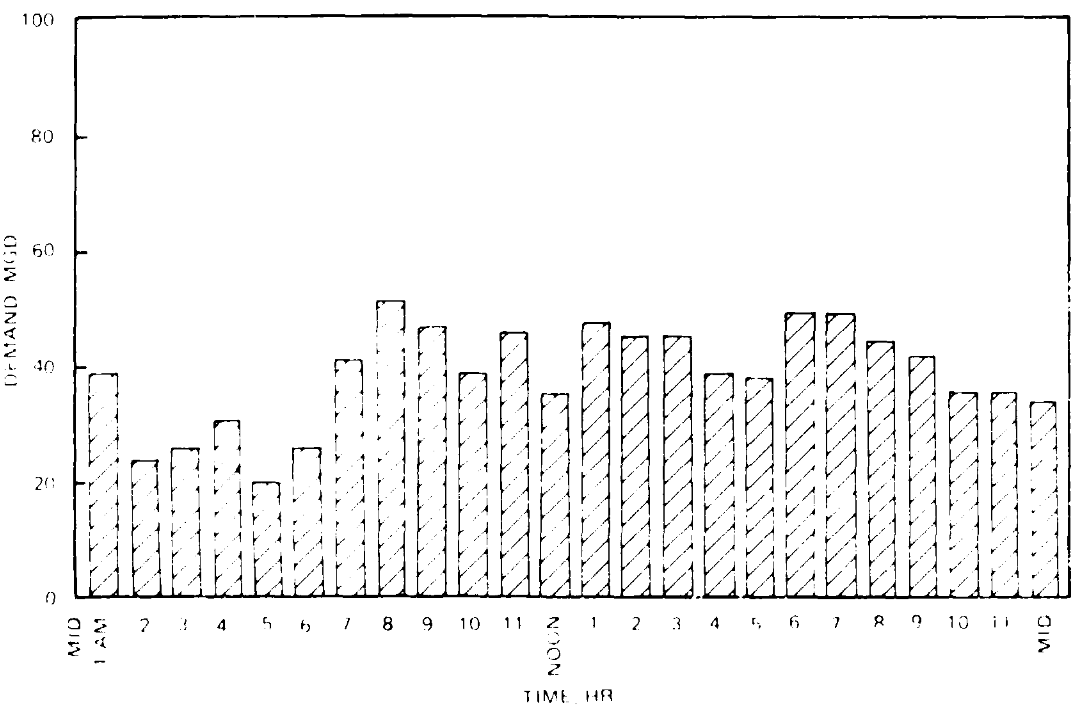


Figure 17. Second high-pressure system demand, Thursday, 20 March 1986

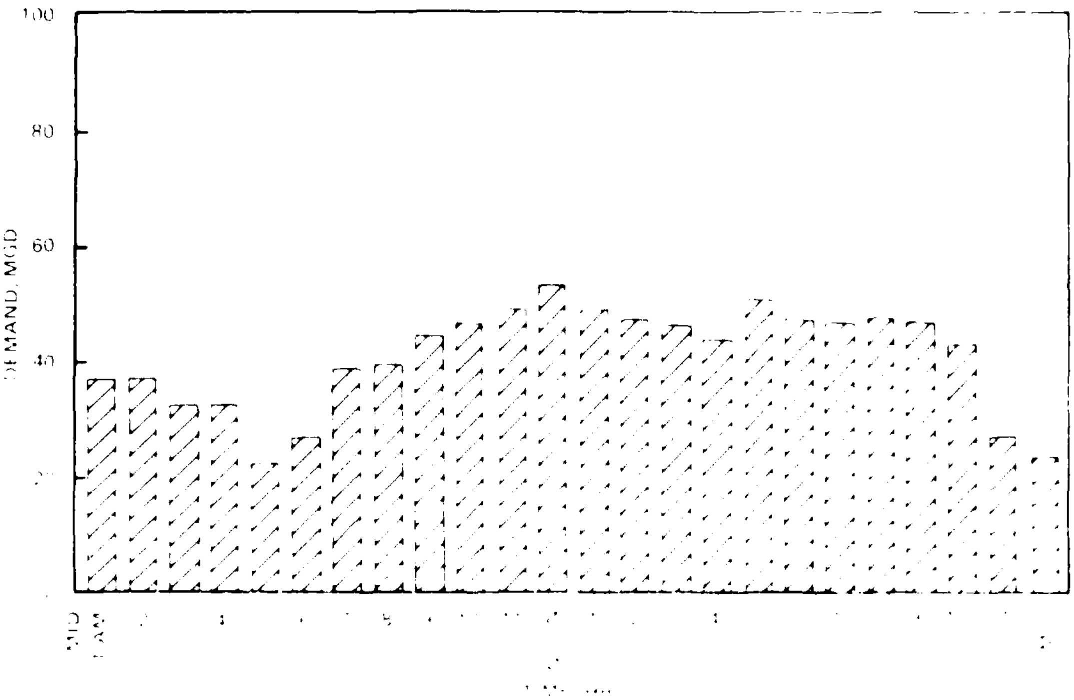


Figure 18. Second high-pressure system demand, Saturday, 29 March 1986

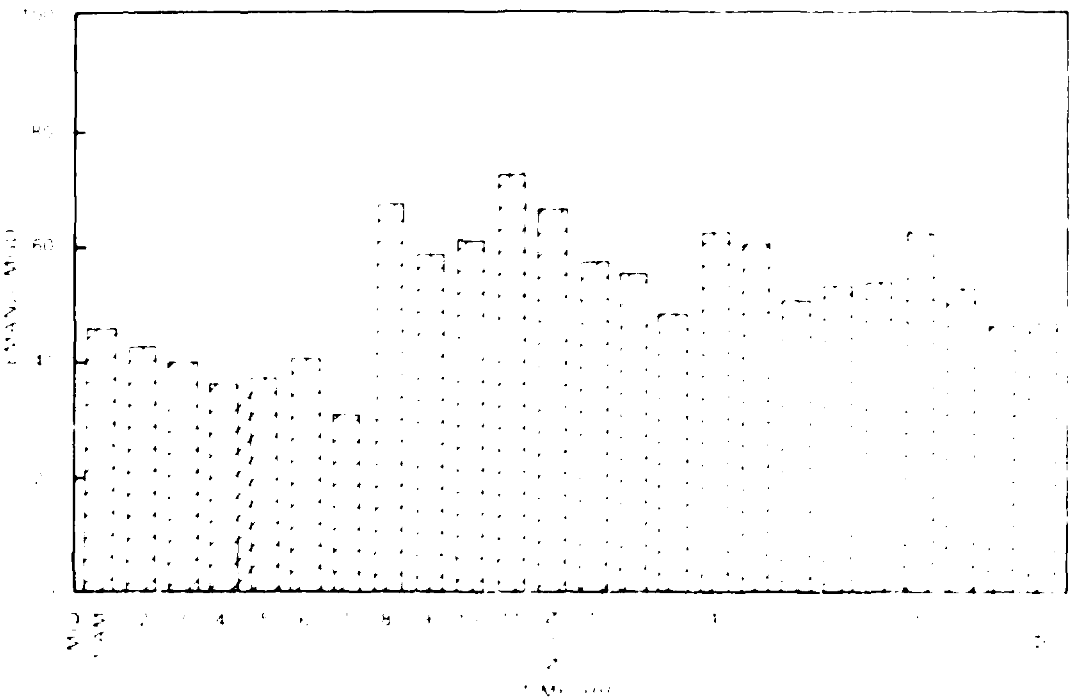


Figure 19. Second high-pressure system demand, Sunday, [illegible]

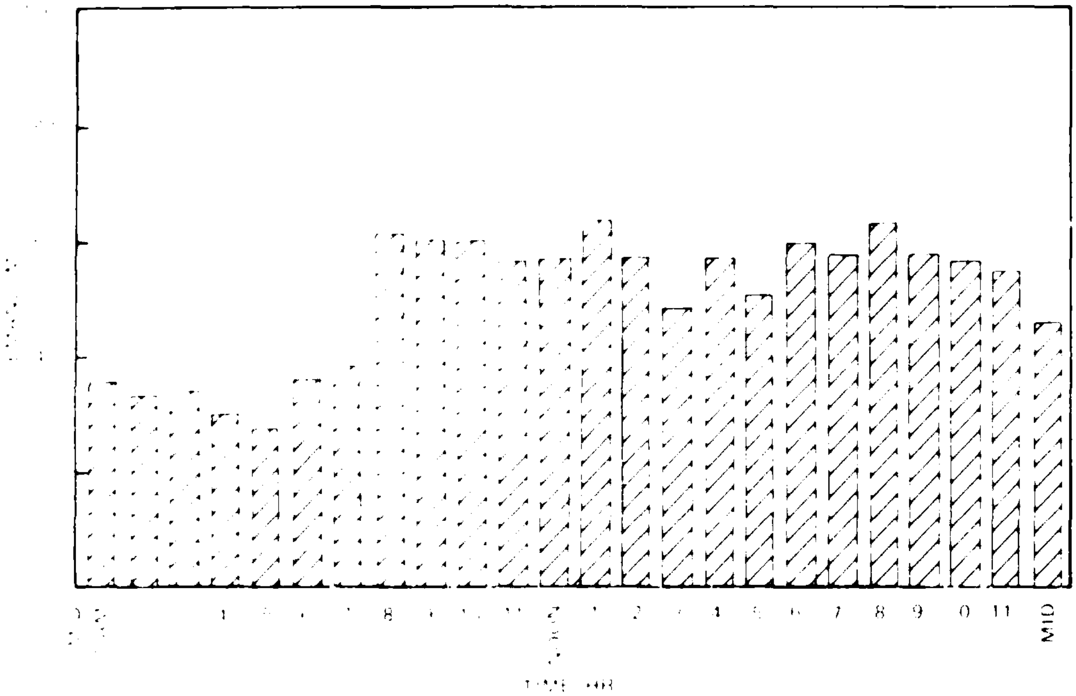
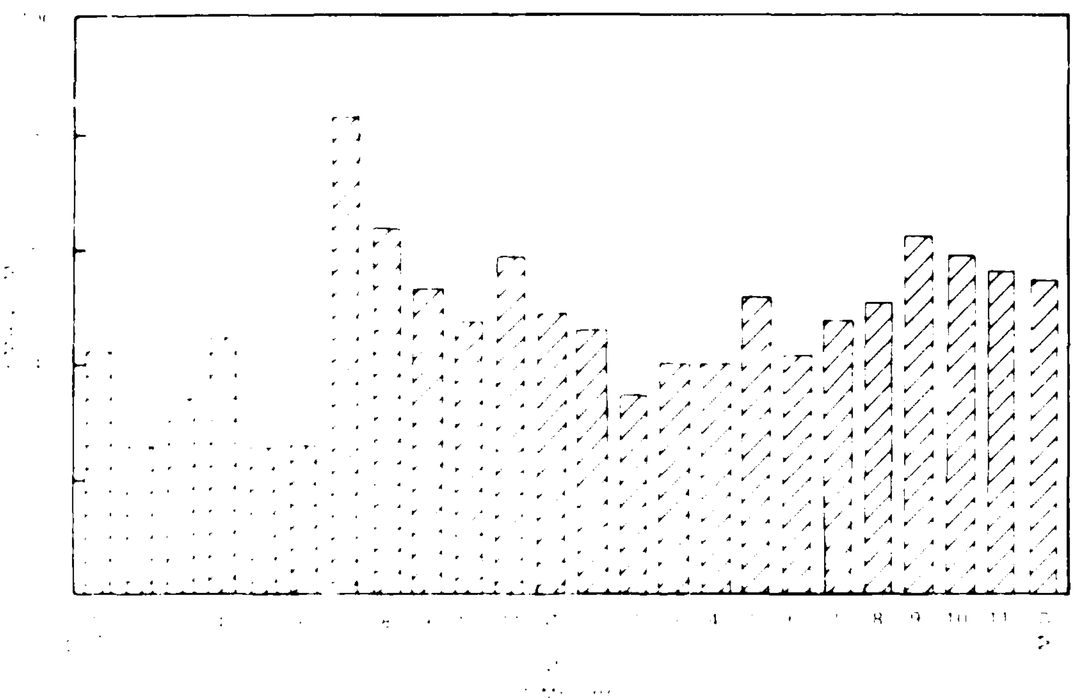


Figure [illegible]. Second high-pressure system demand, Wednesday 11 June 1986



[illegible] high-pressure system demand, Thursday, 20 March 1986

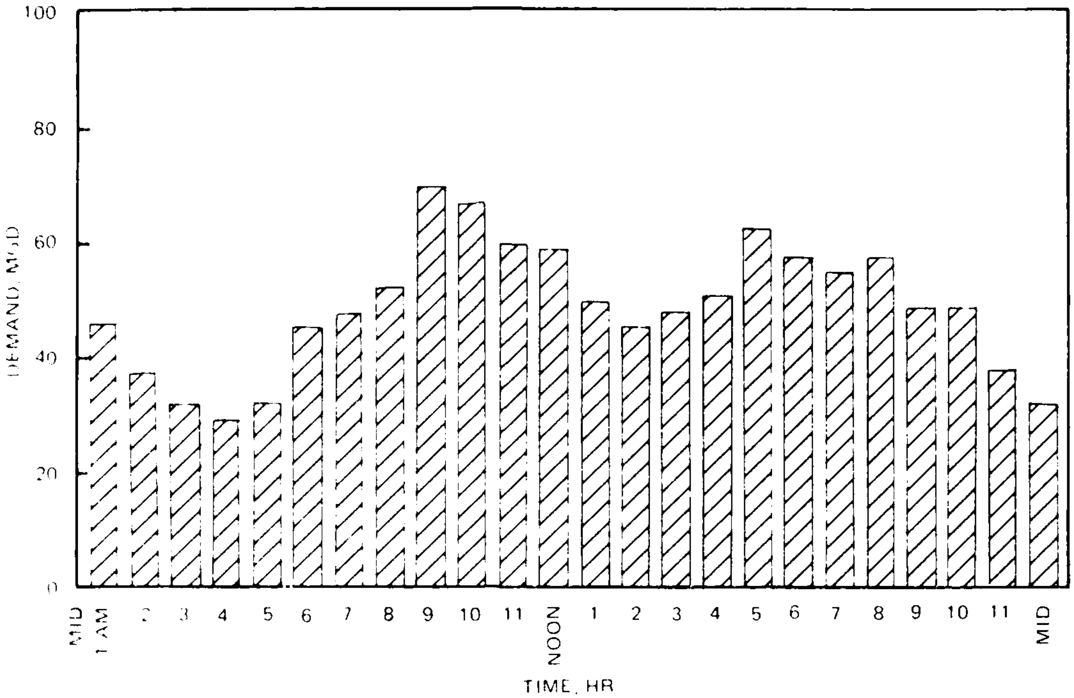


Figure 22. Third high-pressure system demand, Saturday, 29 March 1986

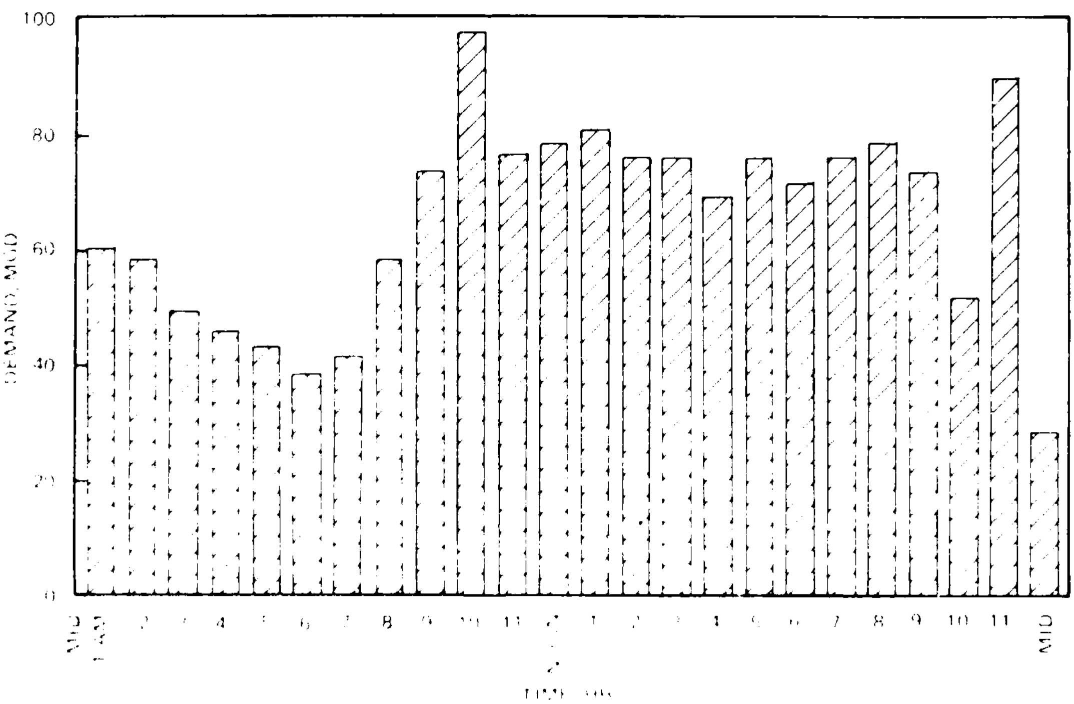


Figure 23. Third high-pressure system demand, Sunday, 8 June 1986

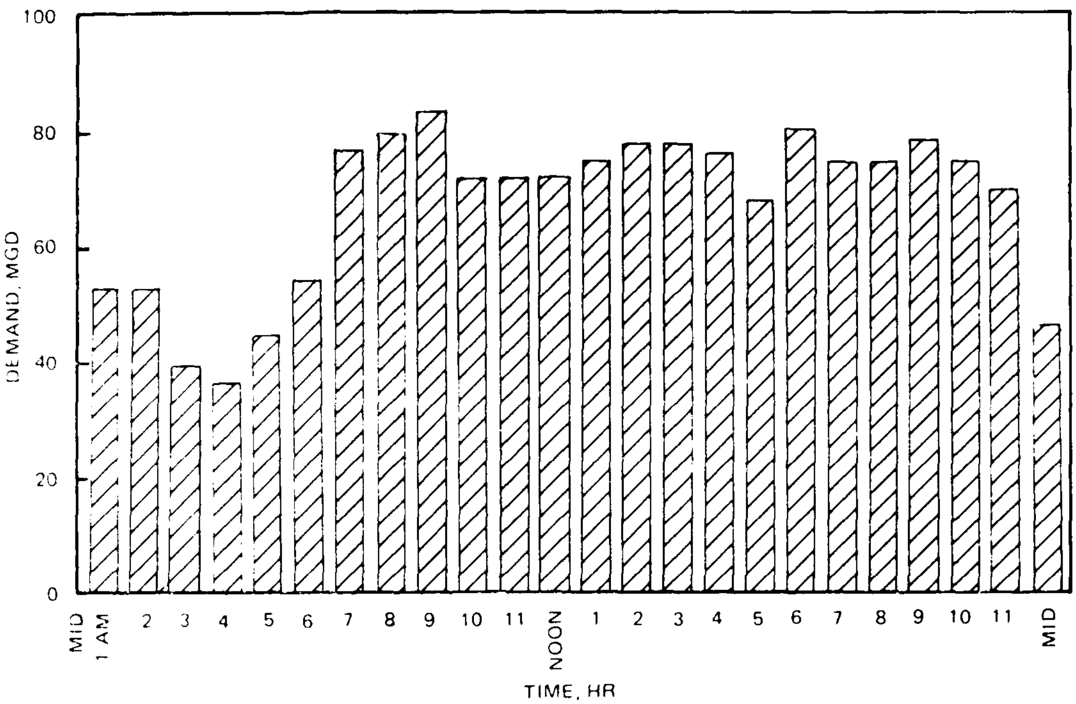


Figure 24. Third high-pressure system demand, Wednesday, 11 June 1986

Network Model Calibration

67. Before the optimal pump operation methodology was applied to each pressure zone, a mathematical model of each system was constructed. Each model was developed based on the network schematics in Figures 11 and 12. The physical parameters associated with each model are shown in Tables 9 and 10.

68. After a mathematical model of each pressure zone was developed, each model was calibrated using operational data for each day. In calibrating each model, a continuous simulation run was made for each day using the observed pumping operating policy and associated system demand data. Each model was calibrated by adjusting selected headloss coefficients. Comparisons between the observed and simulated tank water levels for each day for the final calibration runs are shown in Figures 25-32. As can be seen from the figures, the calibrated models were able to reproduce most of the observed tank water levels very closely for a wide range of system demands and pump operating conditions. The worst case was for the second high-pressure system on 11 June 1986. For this day a maximum error of 2 ft was obtained. Attempts

Table 9

Pipe Characteristics for Second High-Pressure System

Pipe No.	Length ft	Diameter in.	Roughness (C-factor)
1	10,550	30	100
2	9,250	16	100
3	9,000	20	100
4	2,950	24	60
5	800	36	60
6	3,500	36	60
7	11,200	36	100
8	18,300	36	60
9	3,250	36	60
10	4,650	36	60
11	6,900	36	60
12	2,800	36	60
13	14,330	42	60
14	280	42	30
15	500	42	80
16	7,600	36	100
17	700	36	100

Table 10

Pipe Characteristics for Third High-Pressure System

Pipe No.	Length ft	Diameter in.	Roughness (C-factor)
1	580	36	80
2	3,730	36	100
3	6,620	36	100
4	7,050	36	100
5	1,800	24	100
6	9,200	24	80
7	7,130	36	50
8	1,350	36	150
9	500	36	150
10	17,490	48	100
11	2,250	20	100
12	12,480	48	150
13	16,880	48	150

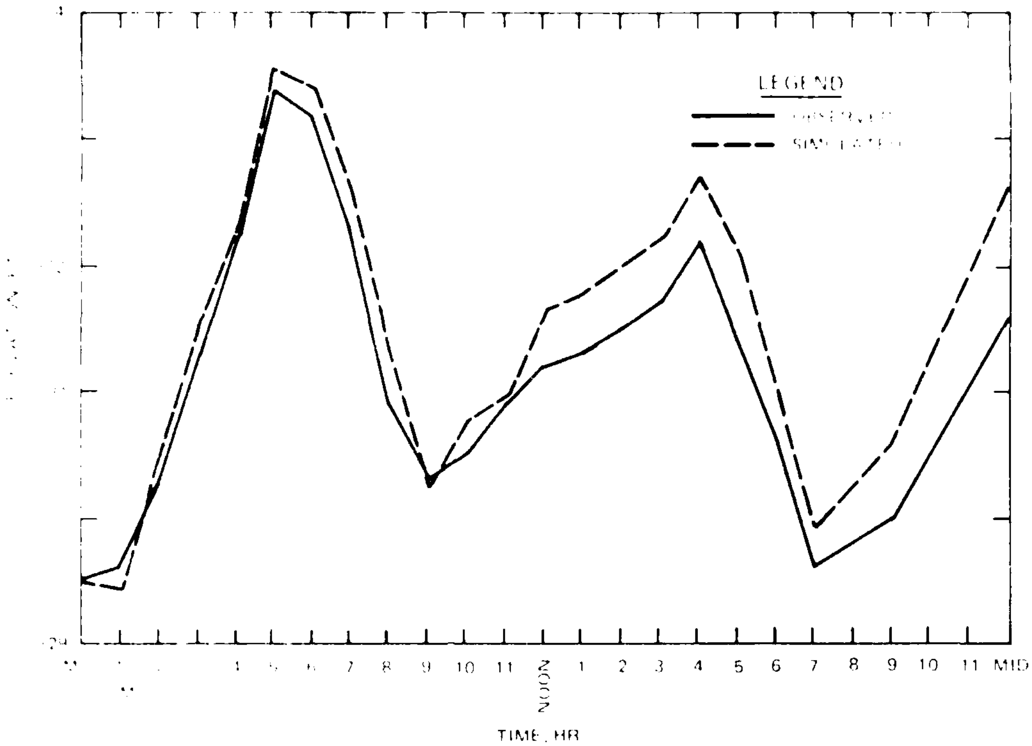


Figure 25. Second high-pressure system calibration results, Thursday, 20 March 1986

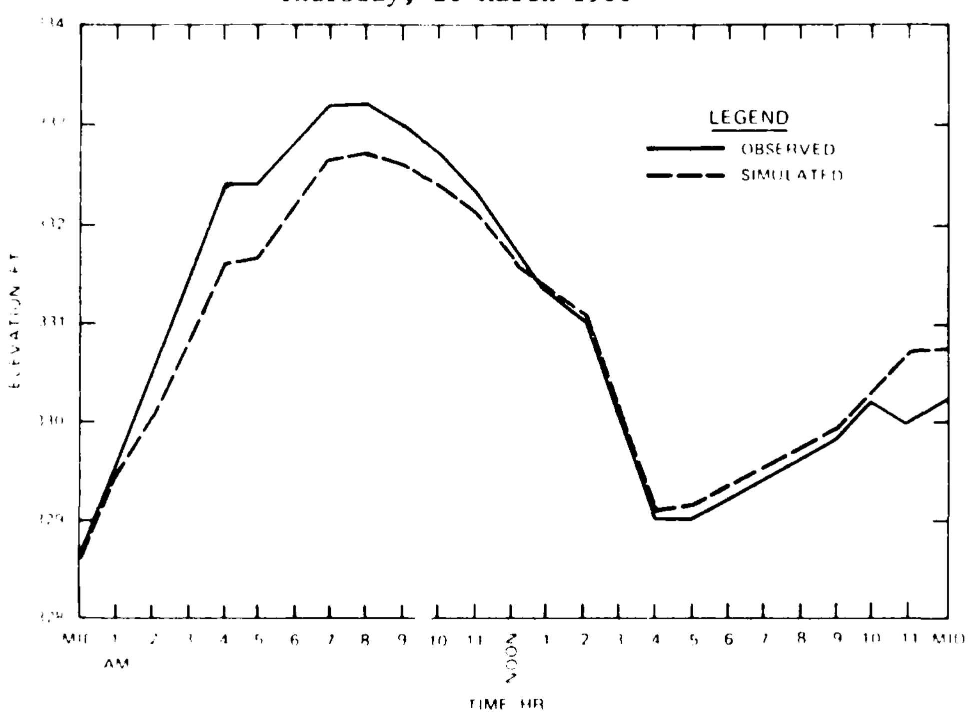


Figure 26. Second high-pressure system calibration results, Saturday, 29 March 1986

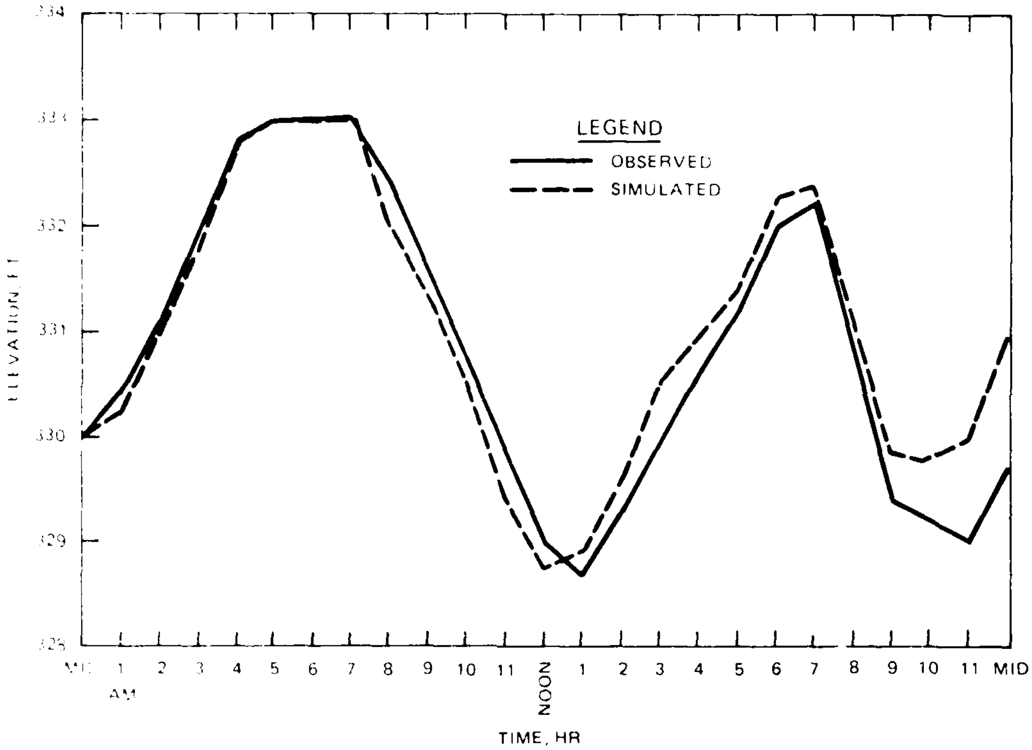


Figure 27. Second high-pressure system calibration results, Sunday, 8 June 1986

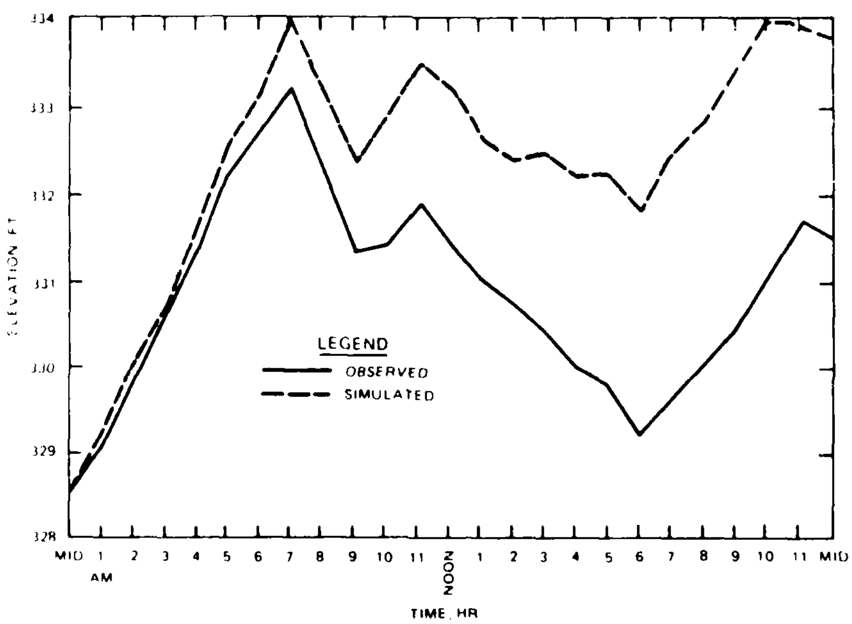


Figure 28. Second high-pressure system calibration results, Wednesday, 11 June 1986

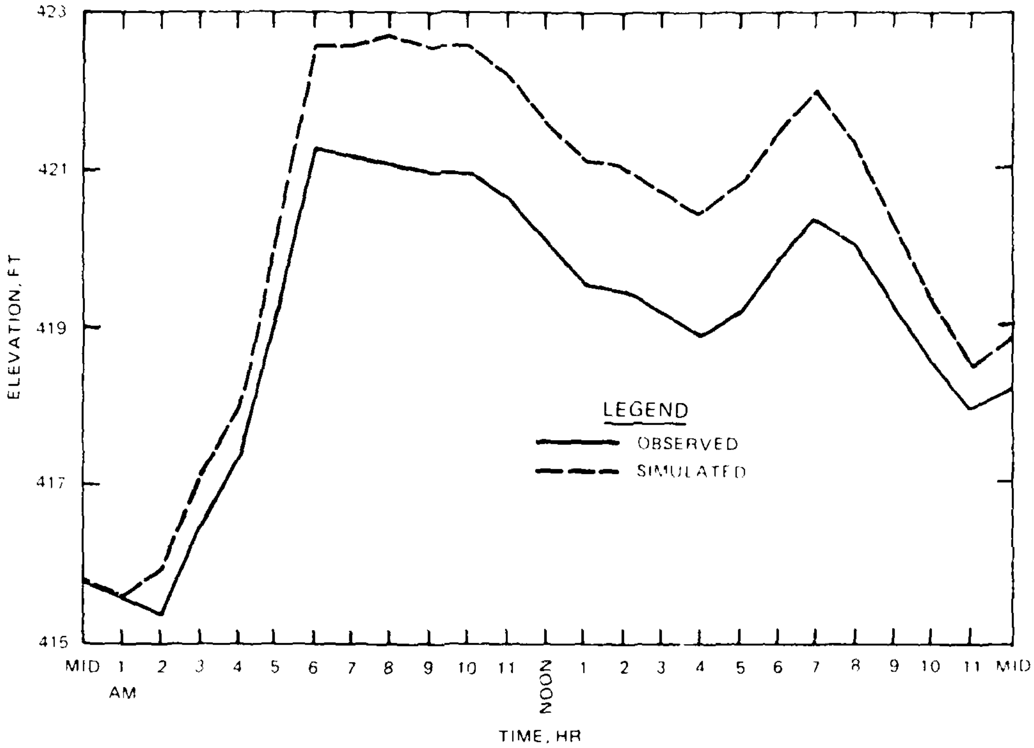


Figure 29. Third high-pressure system calibration results, Thursday, 20 March 1986

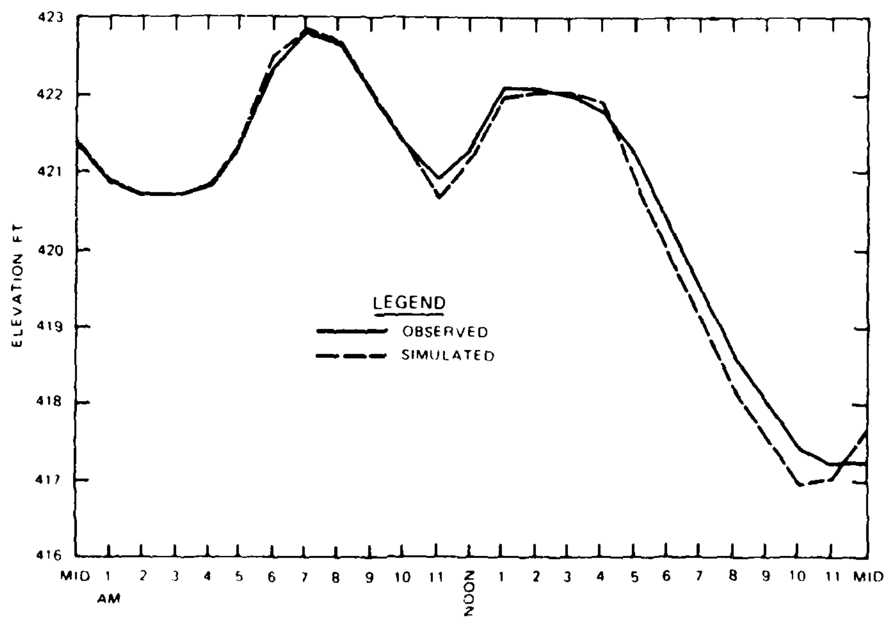


Figure 30. Third high-pressure system calibration results, Saturday, 29 March 1986

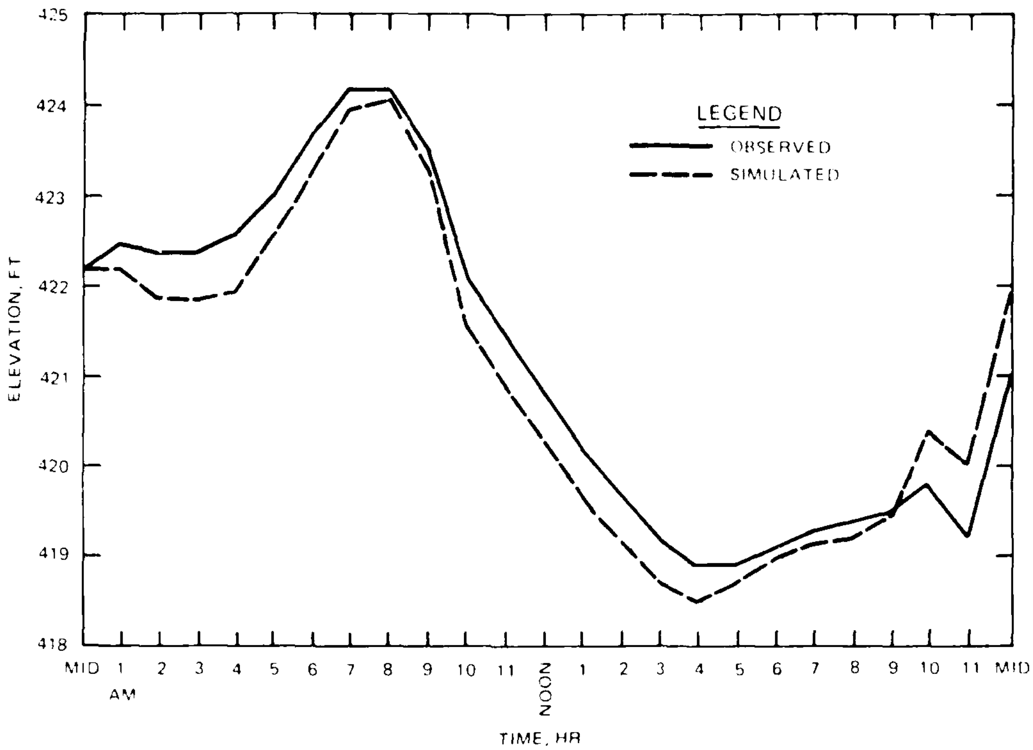


Figure 31. Third high-pressure system calibration results, Saturday, 8 June 1986

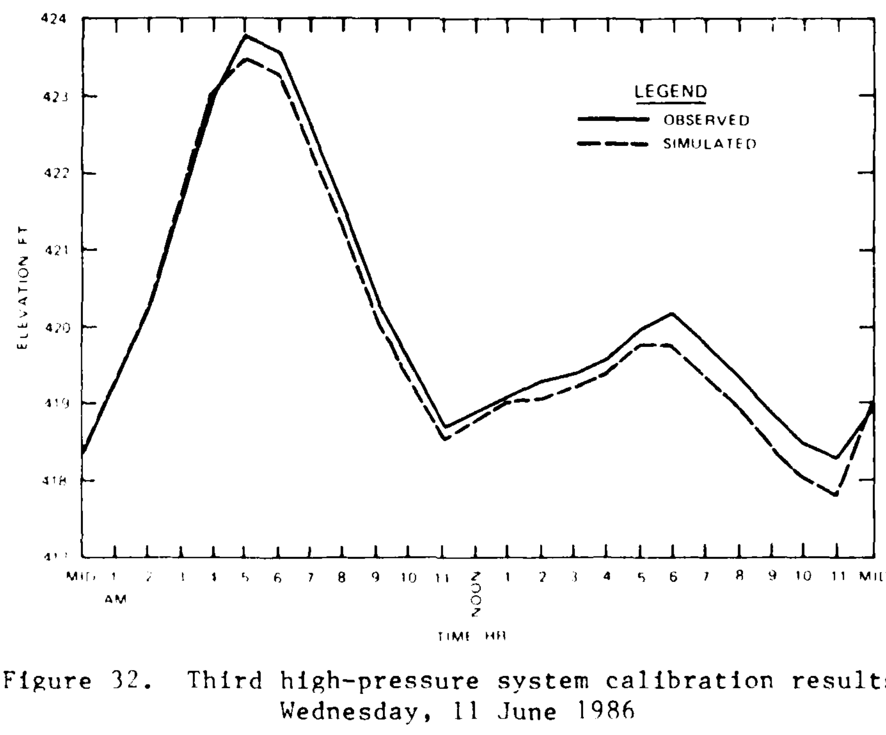


Figure 32. Third high-pressure system calibration results, Wednesday, 11 June 1986

to improve the calibration for this day were limited by some unresolvable discrepancies in the recorded pump flows.

Pump Operation Curves

69. Once a calibrated model of each system was obtained, it was used to develop a set of pump operation curves for each possible pump combination. For the second high-pressure system, 11 different combinations of five pumps are possible. A listing of the possible pump combinations is provided as Table 11. For the third high-pressure system, 27 different combinations of nine pumps are possible. A listing of the possible pump combinations is given as Table 12. For each pump combination, different TLF and TLC curves were obtained for three different system demands. For the second and third high-pressure zones, system demands of 10,000, 35,000, and 60,000 gpm were used. This resulted in three different TLF and TLC curves for each pump combination. Both TLF and TLC curves were developed assuming a constant clearwell level. For Dalecarlia the clearwell level was set at 135 ft while Bryant Street was set at 155 ft. The extra 20 ft of head at Bryant Street is supplied by the McMillan pumping station. The cost of producing this additional head was included in the TLC curves for the Bryant Street station so that both pumping stations would be treated on an equal basis.

70. Each TLF curve was obtained by fitting a quadratic function through three data points. Each data point was obtained by applying the calibrated network model for each pump combination in each system for a specified system demand and tank level. The total flow rate for both pumping stations that results from this simulation was then plotted against the associated tank level. For the second high-pressure system, tank levels of 318, 326.5, and 335 ft were used. For the third system, tank levels of 406, 415, and 424 ft were used.

71. The data points used in constructing the TLF curves for the second high-pressure system are shown in Table 13. A typical set of TLF curves for the second system is shown in Figure 33. The equations corresponding to these curves may be written as follows:

Maximum average demand curve: Q_p = 69,930 - 85.6*h - 0.02076*h^2

Table 11

Pump Combinations for Second High-Pressure System

Pump Combination	Dalecarlia Pumps 7	8	9	Bryant Pumps 7	8
1	X				
2	X	X			
3	X	X	X		
4				X	
5	X			X	
6	X	X		X	
7	X	X	X	X	
8				X	X
9	X			X	X
10	X	X		X	X
11	X	X	X	X	X

Note: X = pump operating.

Table 12

Pump Combinations for Third High-Pressure System

Pump Combination	Dalecarlia Pumps 10	11	12	13	14	15	Bryant Pumps 9	10	11
1	X								
2	X	X							
3	X	X	X						
4	X	X	X	X					
5	X	X	X	X	X				
6	X	X	X	X	X	X			
7							X		
8	X						X		
9	X	X					X		
10	X	X	X				X		
11	X	X	X	X			X		
12	X	X	X	X	X		X		
13	X	X	X	X	X	X	X		
14							X	X	
15	X						X	X	
16	X	X					X	X	
17	X	X	X				X	X	
18	X	X	X	X			X	X	
19	X	X	X	X	X		X	X	
20	X	X	X	X	X	X	X	X	
21							X	X	X
22	X						X	X	X
23	X	X					X	X	X
24	X	X	X				X	X	X
25	X	X	X	X			X	X	X
26	X	X	X	X	X		X	X	X
27	X	X	X	X	X	X	X	X	X

Note: X = pump operating.

Table 13

Data Points for TLF Curves for Second High-Pressure System

Pump Combination	Tank Level ft	Flow Rate gpm	Tank Level ft	Flow Rate gpm	Tank Level ft	Flow Rate gpm
1	335.	19,224.	326.5	19,795.	318.	20,373.
	335.	16,898.	326.5	17,574.	318.	18,240.
	335.	15,433.	326.5	16,032.	318.	16,629.
2	335.	33,194.	326.5	34,234.	318.	35,250.
	335.	29,289.	326.5	30,307.	318.	31,240.
	335.	24,304.	326.5	25,465.	318.	26,525.
3	335.	42,368.	326.5	43,529.	318.	44,651.
	335.	35,627.	326.5	37,036.	318.	38,350.
	335.	27,147.	326.5	28,679.	318.	30,107.
4	335.	21,734.	326.5	21,988.	318.	22,232.
	335.	18,225.	326.5	18,729.	318.	19,205.
	335.	15,312.	326.5	15,912.	318.	16,464.
5	335.	38,937.	326.5	39,781.	318.	40,622.
	335.	34,355.	326.5	35,586.	318.	36,770.
	335.	29,444.	326.5	30,599.	318.	31,690.
6	335.	52,551.	326.5	53,906.	318.	55,239.
	335.	46,317.	326.5	47,811.	318.	49,198.
	335.	36,513.	326.5	38,204.	318.	39,771.
7	335.	61,584.	326.5	63,183.	318.	64,656.
	335.	51,647.	326.5	53,506.	318.	55,245.
	335.	38,518.	326.5	40,510.	318.	42,373.
8	335.	33,302.	326.5	34,228.	318.	35,136.
	335.	27,680.	326.5	28,562.	318.	29,386.
	335.	18,954.	326.5	19,868.	318.	20,731.
9	335.	49,515.	326.5	51,082.	318.	52,656.
	335.	43,566.	326.5	45,076.	318.	46,520.
	335.	32,445.	326.5	33,870.	318.	35,206.
10	335.	63,253.	326.5	65,354.	318.	67,371.
	335.	54,547.	326.5	56,311.	318.	57,948.
	335.	38,626.	326.5	40,517.	318.	42,294.
11	335.	71,740.	326.5	73,933.	318.	76,014.
	335.	59,195.	326.5	61,277.	318.	63,206.
	335.	40,349.	326.5	42,497.	318.	44,502.

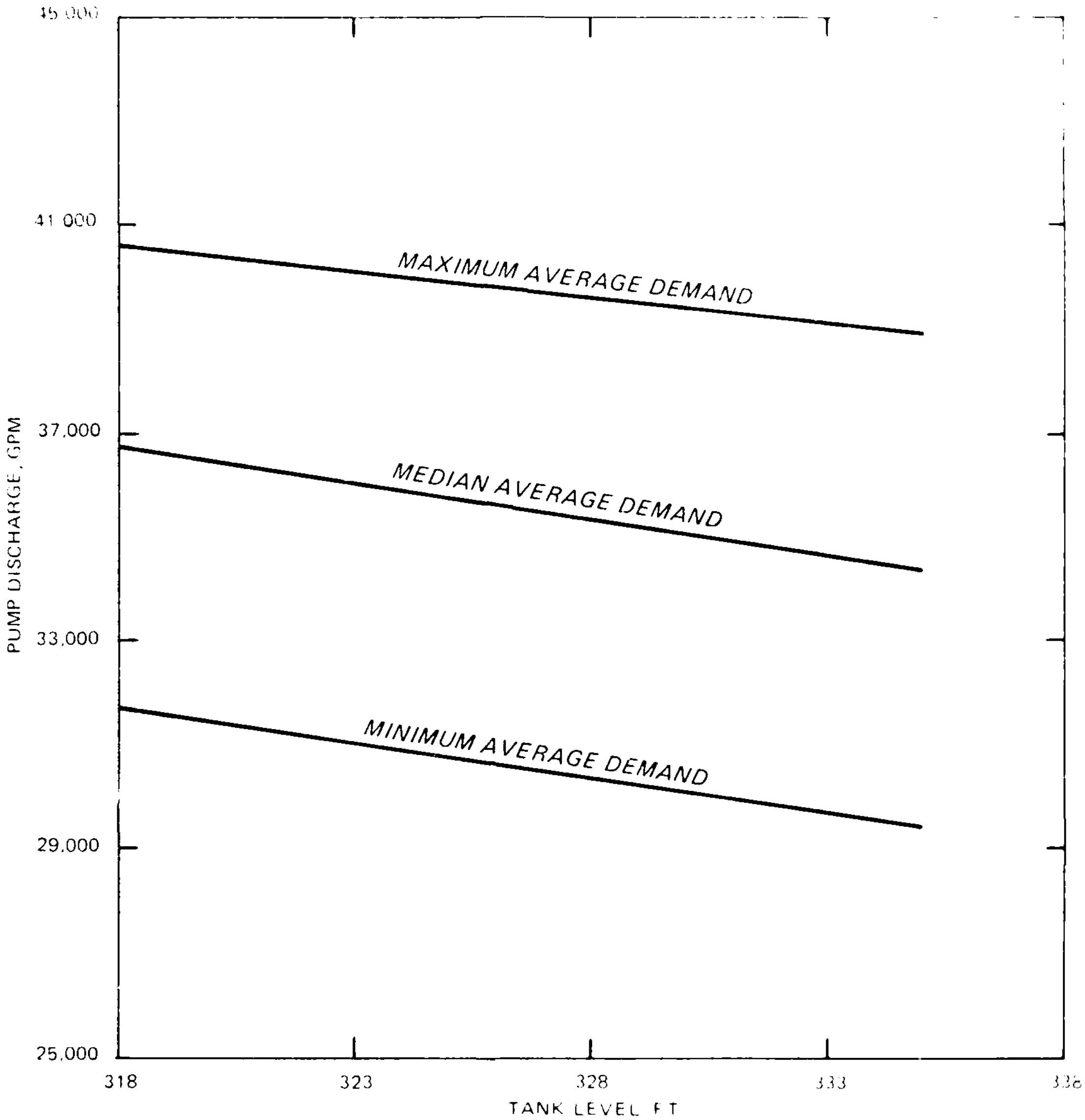


Figure 33. Typical set of TLF curves for second high-pressure system

Median average demand curve: $Q_p = 47{,}295 + 70.3*h - 0.32526*h^2$

Minimum average demand curve: $Q_p = 26{,}521 + 157.1*h - 0.44291*h^2$

where

Q_p = pump discharge, gpm

h = tank water level, ft

The data points used in constructing the TLF curves for the third high-pressure system are shown in Table 14. A typical set of TLF curves for the

Table 1[illegible]

Data Points for [illegible] Curves for Third High-Pressure System

Pump Combination	Tank Level	Flow Rate gpm	Tank Level	Flow Rate gpm	Tank Level	Flow Rate gpm
[illegible]	[illegible]	[illegible]	415.	[illegible]	406.	24,418.
	[illegible]	[illegible]	415.	22,918.	406.	23,916.
	424.	[illegible]	415.	22,455.	406.	23,431.
[illegible]	424.	43,229.	415.	45,068.	406.	46,884.
	424.	41,989.	415.	43,793.	406.	45,587.
	424.	40,082.	415.	41,785.	406.	43,477.
3	424.	61,070.	415.	63,558.	406.	66,028.
	424.	58,566.	415.	60,935.	406.	63,202.
	424.	54,251.	415.	56,772.	406.	59,090.
[illegible]	424.	75,629.	415.	78,652.	406.	81,450.
	424.	70,687.	415.	73,778.	406.	76,644.
	424.	63,648.	415.	66,987.	406.	70,056.
5	424.	85,990.	415.	89,628.	406.	92,985.
	424.	78,658.	415.	82,408.	406.	85,849.
	424.	69,300.	415.	[illegible]	406.	76,937.
[illegible]	424.	92,832.	415.	96,967.	406.	100,819.
	424.	83,599.	415.	87,823.	406.	91,749.
	424.	[illegible]	415.	76,915.	406.	81,027.
[illegible]	424.	[illegible]	415.	12,234.	406.	12,562.
	424.	11,642.	415.	11,969.	406.	12,296.
	424.	[illegible]	415.	11,766.	406.	12,089.
[illegible]	424.	33,984.	[illegible]	[illegible]	406.	36,607.
	424.	[illegible]	[illegible]	34,780.	406.	36,105.
	424.	[illegible]	[illegible]	34,018.	406.	35,299.
[illegible]	[illegible]	[illegible]	415.	56,919.	406.	59,102.
	[illegible]	[illegible]	[illegible]	55,438.	406.	57,534.
	[illegible]	[illegible]	[illegible]	[illegible]	406.	54,911.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	406.	78,093.
	[illegible]	[illegible]	[illegible]	[illegible]	406.	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	406.	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	406.	87,761.
	[illegible]	[illegible]	[illegible]	[illegible]	406.	[illegible]

(Continued)

Table 14 (Continued)

Pump Combination	Tank Level	Flow Rate gpm	Tank Level	Flow Rate gpm	Tank Level	Flow Rate gpm
12	424.	96,623.	415.	100,594.	406.	104,254.
	424.	88,805.	415.	92,821.	406.	96,549.
	424.	78,765.	415.	83,023.	406.	86,960.
13	424.	103,200.	415.	107,627.	406.	111,711.
	424.	93,441.	415.	97,943.	406.	102,117.
	424.	81,715.	415.	86,391.	406.	90,745.
14	424.	22,167.	415.	22,754.	406.	23,329.
	424.	21,491.	415.	22,111.	406.	22,686.
	424.	20,825.	415.	21,479.	406.	22,088.
15	424.	44,058.	415.	45,643.	406.	47,211.
	424.	43,257.	415.	44,882.	406.	46,450.
	424.	41,825.	415.	43,432.	406.	44,978.
16	424.	64,900.	415.	67,360.	406.	69,762.
	424.	62,836.	415.	65,216.	406.	67,517.
	424.	59,576.	415.	61,899.	406.	64,144.
17	424.	82,414.	415.	85,415.	406.	88,371.
	424.	78,573.	415.	81,562.	406.	84,326.
	424.	72,679.	415.	75,888.	406.	78,821.
18	424.	96,183.	415.	99,725.	406.	103,019.
	424.	89,798.	415.	93,487.	406.	96,916.
	424.	81,122.	415.	85,093.	406.	88,782.
19	424.	105,704.	415.	109,889.	406.	113,776.
	424.	97,012.	415.	101,321.	406.	105,315.
	424.	86,018.	415.	90,593.	406.	94,857.
20	424.	111,982.	415.	116,635.	406.	120,945.
	424.	101,383.	415.	106,161.	406.	110,589.
	424.	88,737.	415.	93,729.	406.	98,377.
21	424.	31,739.	415.	32,800.	406.	33,804.
	424.	30,205.	415.	31,316.	406.	32,371.
	424.	28,603.	415.	29,760.	406.	30,848.
22	424.	53,501.	415.	55,576.	406.	57,613.
	424.	51,831.	415.	53,907.	406.	55,932.
	424.	49,203.	415.	51,284.	406.	53,286.

(Continued)

(Sheet 2 of 3)

Table 14 (Concluded)

Pump Combination	Tank Level	Flow Rate gpm	Tank Level	Flow Rate gpm	Tank Level	Flow Rate gpm
23	424.	74,258.	415.	77,135.	406.	79,944.
	424.	70,986.	415.	73,784.	406.	76,503.
	424.	66,406.	415.	69,193.	406.	71,861.
24	424.	91,360.	415.	94,761.	406.	98,089.
	424.	86,182.	415.	89,600.	406.	92,787.
	424.	78,962.	415.	82,600.	406.	85,973.
25	424.	104,621.	415.	108,598.	406.	112,270.
	424.	96,916.	415.	101,045.	406.	104,852.
	424.	86,927.	415.	91,332.	406.	95,421.
26	424.	113,708.	415.	118,254.	406.	122,524.
	424.	103,769.	415.	108,445.	406.	112,822.
	424.	91,507.	415.	96,487.	406.	101,113.
27	424.	119,608.	415.	124,662.	406.	129,349.
	424.	107,870.	415.	112,992.	406.	117,786.
	424.	94,022.	415.	99,392.	406.	104,367.

(Sheet 3 of 3)

third system is shown in Figure 34. The equations corresponding to these curves may be written as follows:

$$\text{Maximum average demand curve:}\quad Q_p = 101{,}083 - 171.3*h + 0.03086*h\hat{}2$$

$$\text{Median average demand curve:}\quad Q_p = 88{,}597 - 111.7*h - 0.04321*h\hat{}2$$

$$\text{Minimum average demand curve:}\quad Q_p = 94{,}126 - 147.4*h + 0.00617*h\hat{}2$$

2. Similar to the TLF curves, the TLC curves were obtained by fitting a quadratic function through three different data points. As before, each data point was obtained as a result of applying the calibrated network model for each system for a specified system demand and tank water level. In this case, the resulting pump head, pump efficiency, and flow rate for each pump were all used to calculate a total unit cost for the total resulting flow rate. As before, this unit cost was plotted against the specified tank water level for each system demand. The data points used in constructing the TLC curves for the second high-pressure system are shown in Table 15. A typical set of TLC curves for the second system is shown in Figure 35. The equations corresponding to these curves may be written as follows:

$$\text{Maximum average demand curve:}\quad UC = 1.276 - 0.003954*h + 0.00007*h\hat{}2$$

$$\text{Median average demand curve:}\quad UC = 1.644 - 0.006325*h + 0.00001*h\hat{}2$$

$$\text{Minimum average demand curve:}\quad UC = 1.698 - 0.006828*h + 0.00001*h\hat{}2$$

where UC is the unit cost in kilowatt-hours per 1,000 gal. The data points used in constructing the TLC curves for the third high-pressure system are shown in Table 16. A typical set of TLC curves for the third system is shown in Figure 36. The equations corresponding to these curves may be written as follows:

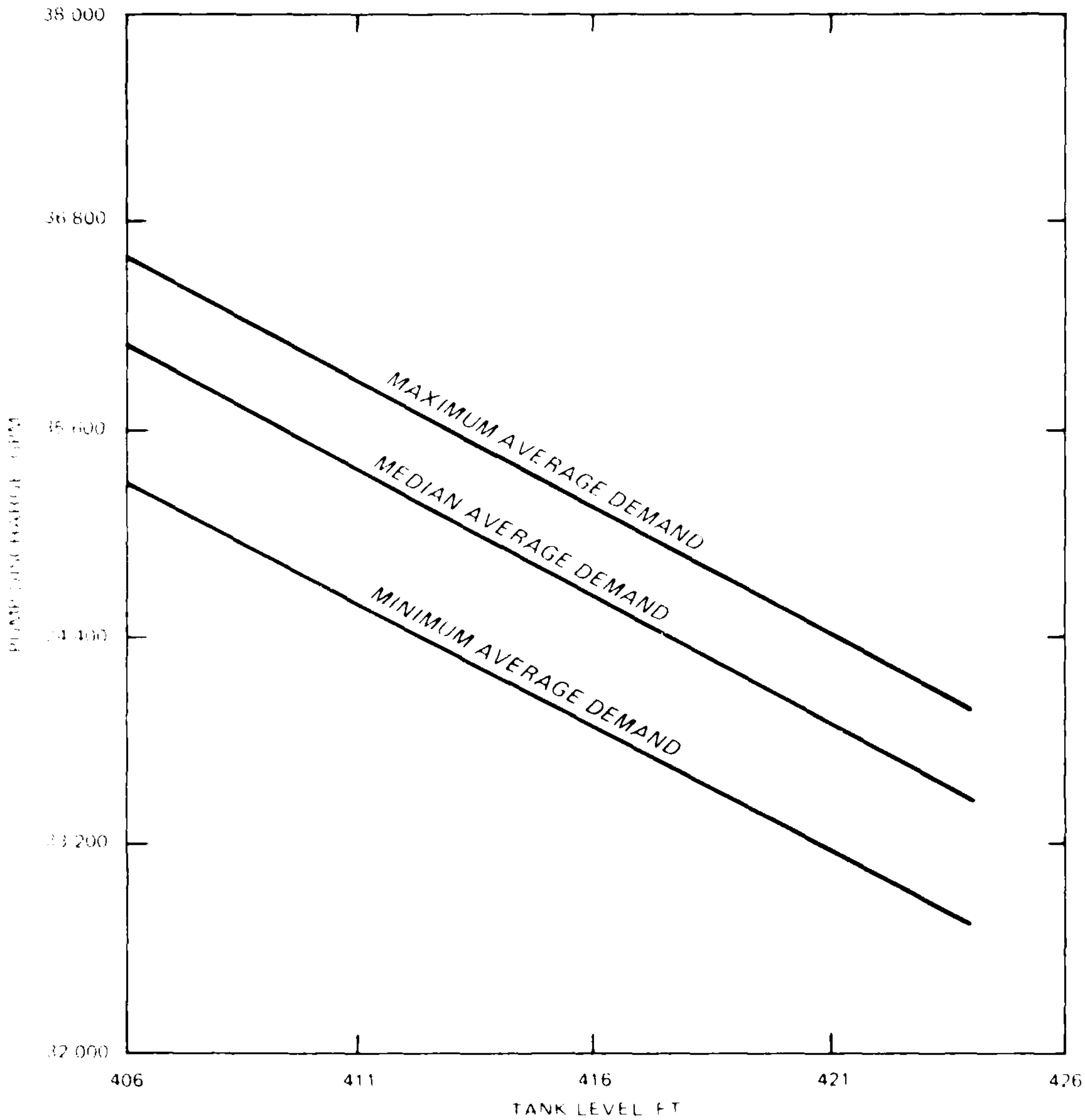


Figure 34. Typical set of TLF curves for third high-pressure system

Maximum average demand curve: UC = 3.516 - 0.012683*h + 0.00002*h^2

Median average demand curve: UC = 3.861 - 0.014533*h + 0.00002*h^2

Minimum average demand curve: UC = 3.273 - 0.011821*h + 0.00002*h^2

For a given pump combination, system demand, and average tank water level, the PCP program uses the corresponding TLF and TLC curves to obtain a required

Table 15

Data Points for TLC Curves for Second High-Pressure System

Pump Combination	Tank Level	Unit Cost*	Tank Level	Unit Cost*	Tank Level	Unit Cost*
1	335.	0.7674	326.5	0.7713	318.	0.7792
	335.	0.7816	326.5	0.7731	318.	0.7681
	335.	0.8112	326.5	0.7972	318.	0.7858
2	335.	0.7864	326.5	0.7785	318.	0.7726
	335.	0.8320	326.5	0.8186	318.	0.8065
	335.	0.9028	326.5	0.8854	318.	0.8702
3	335.	0.8470	326.5	0.8362	318.	0.8260
	335.	0.9116	326.5	0.8968	318.	0.8839
	335.	1.0311	326.5	1.0041	318.	0.9816
4	335.	0.6744	326.5	0.6654	318.	0.6555
	335.	0.7443	326.5	0.7380	318.	0.7314
	335.	0.7793	326.5	0.7714	318.	0.7647
5	335.	0.7281	326.5	0.7228	318.	0.7185
	335.	0.7677	326.5	0.7592	318.	0.7523
	335.	0.8088	326.5	0.7968	318.	0.7866
6	335.	0.7607	326.5	0.7515	318.	0.7434
	335.	0.8069	326.5	0.7965	318.	0.7865
	335.	0.8818	326.5	0.8664	318.	0.8533
7	335.	0.8068	326.5	0.7971	318.	0.7878
	335.	0.8688	326.5	0.8571	318.	0.8466
	335.	0.9816	326.5	0.9593	318.	0.9405
8	335.	0.7625	326.5	0.7571	318.	0.7502
	335.	0.8031	326.5	0.7952	318.	0.7884
	335.	0.9509	326.5	0.9267	318.	0.9063
9	335.	0.7715	326.5	0.7657	318.	0.7610
	335.	0.8014	326.5	0.7921	318.	0.7849
	335.	0.9167	326.5	0.8955	318.	0.8773
10	335.	0.7879	326.5	0.7790	318.	0.7715
	335.	0.8327	326.5	0.8221	318.	0.8126
	335.	0.9796	326.5	0.9557	318.	0.9353
11	335.	0.8255	326.5	0.8165	318.	0.8083
	335.	0.8874	326.5	0.8752	318.	0.8647
	335.	1.0748	326.5	1.0445	318.	1.0189

* Expressed in kilowatt-hours per 1,000 gal at a cost of $0.01 per kilowatt-hour.

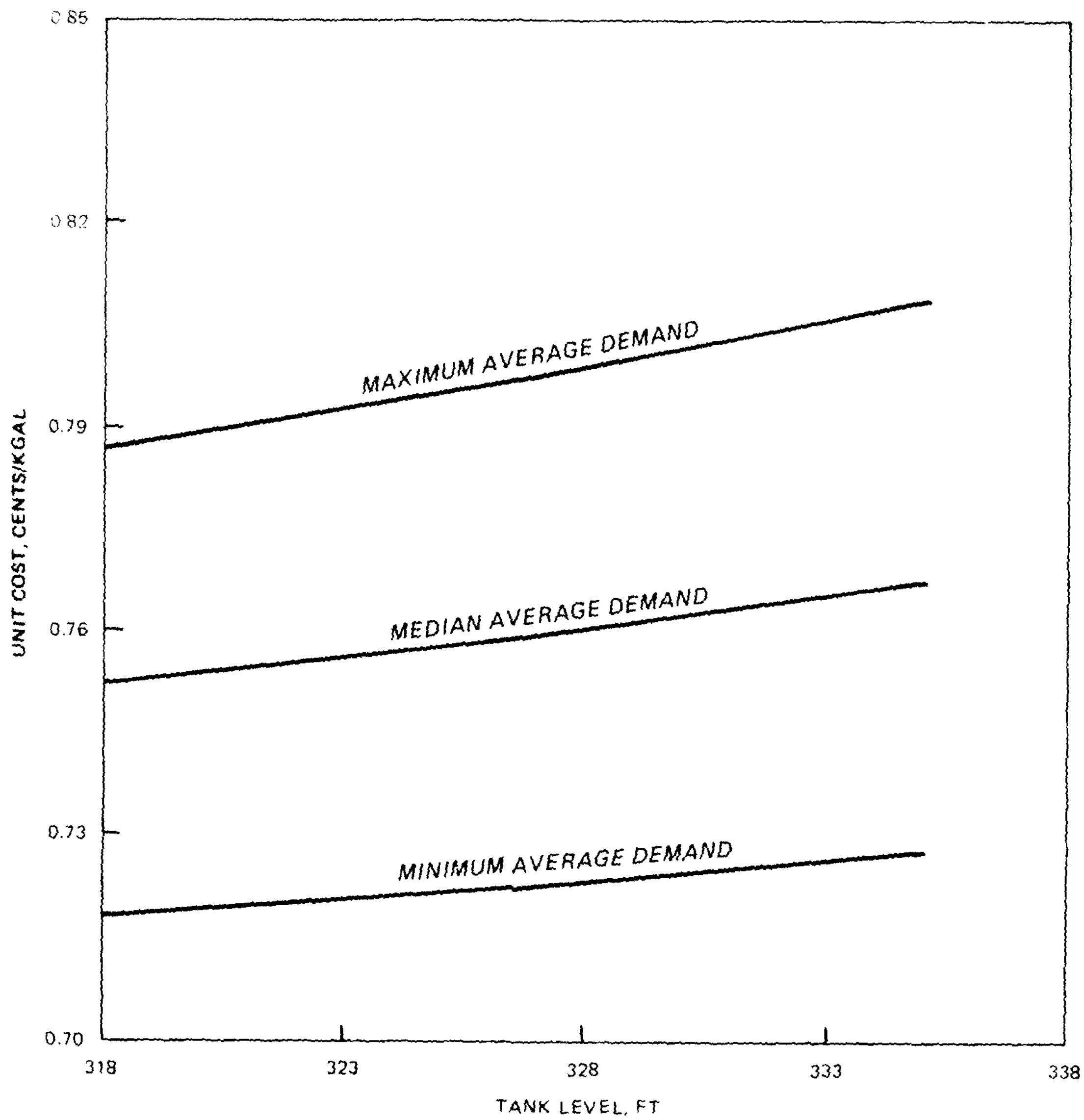


Figure 35. Typical set of TLC curves for second high-pressure system

pump discharge and unit cost. The total cost of a particular pump combination is obtained by multiplying the unit cost by the required pump discharge. Both the unit cost and the required pump discharge are obtained by fitting a quadratic equation through three points obtained from the three TLF and TLC curves for a given average tank water level. The unit cost and required pump discharge are then expressed as a function of the system demand. For the typical set of TLF curves shown in Figures 33 and 34 and tank water levels of 330 and 419 ft, the following pump discharge equations are obtained:

Table 16

Data Points for TLC Curves for Third High-Pressure System

Pump Combination	Tank Level	Unit Cost*	Tank Level	Unit Cost*	Tank Level	Unit Cost*
1	424.	1.0478	415.	1.0182	406.	0.9893
	424.	1.0640	415.	1.0335	406.	1.0040
	424.	1.0779	415.	1.0476	406.	1.0183
2	424.	1.0736	415.	1.0453	406.	1.0179
	424.	1.0933	415.	1.0647	406.	1.0372
	424.	1.1243	415.	1.0966	406.	1.0698
3	424.	1.1140	415.	1.0872	406.	1.0613
	424.	1.1408	415.	1.1154	406.	1.0908
	424.	1.1831	415.	1.1590	406.	1.1354
4	424.	1.1596	415.	1.1365	406.	1.1138
	424.	1.1943	415.	1.1729	406.	1.1520
	424.	1.2386	415.	1.2183	406.	1.1985
5	424.	1.2068	415.	1.1874	406.	1.1685
	424.	1.2429	415.	1.2249	406.	1.2074
	424.	1.2851	415.	1.2677	406.	1.2510
6	424.	1.2490	415.	1.2327	406.	1.2169
	424.	1.2835	415.	1.2680	406.	1.2532
	424.	1.3227	415.	1.3073	406.	1.2928
7	424.	1.2967	415.	1.3272	406.	1.3665
	424.	1.2798	415.	1.3025	406.	1.3338
	424.	1.2688	415.	1.2875	406.	1.3248
8	424.	1.1345	415.	1.1228	406.	1.1138
	424.	1.1389	415.	1.1252	406.	1.1146
	424.	1.1460	415.	1.1312	406.	1.1190
9	424.	1.1202	415.	1.1020	406.	1.0864
	424.	1.1347	415.	1.1165	406.	1.1003
	424.	1.1598	415.	1.1411	406.	1.1241
10	424.	1.1422	415.	1.1234	406.	1.1064
	424.	1.1663	415.	1.1480	406.	1.1314
	424.	1.2019	415.	1.1842	406.	1.1673
11	424.	1.1785	415.	1.1615	406.	1.1457
	424.	1.2092	415.	1.1931	406.	1.1781
	424.	1.2470	415.	1.2314	406.	1.2168

(Continued)

* Expressed in kilowatt-hours per 1,000 gal at a cost of $0.01 per kilowatt-hour.

(Sheet 1 of 3)

Table 16 (Continued)

Pump Combination	Tank Level	Unit Cost	Tank Level	Unit Cost	Tank Level	Unit Cost
12	424.	1.2188	415.	1.2040	406.	1.1905
	424.	1.2502	415.	1.2365	406.	1.2236
	424.	1.2862	415.	1.2728	406.	1.2601
13	424.	1.2553	415.	1.2428	406.	1.2315
	424.	1.2853	415.	1.2733	406.	1.2625
	424.	1.3183	415.	1.3066	406.	1.2957
14	424.	1.2552	415.	1.2662	406.	1.2812
	424.	1.2449	415.	1.2542	406.	1.2646
	424.	1.2369	415.	1.2445	406.	1.2537
15	424.	1.1548	415.	1.1438	406.	1.1344
	424.	1.1545	415.	1.1431	406.	1.1320
	424.	1.1601	415.	1.1479	406.	1.1363
16	424.	1.1523	415.	1.1452	406.	1.1234
	424.	1.1482	415.	1.1323	406.	1.1169
	424.	1.1697	415.	1.1535	406.	1.1383
17	424.	1.1531	415.	1.1363	406.	1.1201
	424.	1.1749	415.	1.1588	406.	1.1432
	424.	1.2063	415.	1.1906	406.	1.1755
18	424.	1.1854	415.	1.1701	406.	1.1551
	424.	1.2129	415.	1.1984	406.	1.1845
	424.	1.2465	415.	1.2326	406.	1.2194
19	424.	1.2218	415.	1.2087	406.	1.1955
	424.	1.2498	415.	1.2374	406.	1.2256
	424.	1.2817	415.	1.2697	406.	1.2582
20	424.	1.2549	415.	1.2436	406.	1.2328
	424.	1.2815	415.	1.2708	406.	1.2608
	424.	1.3107	415.	1.3001	406.	1.2903
21	424.	1.5080	415.	1.5947	406.	1.7153
	424.	1.4268	415.	1.4807	406.	1.5555
	424.	1.3761	415.	1.4093	406.	1.4556
22	424.	1.3132	415.	1.3467	406.	1.3992
	424.	1.2732	415.	1.2898	406.	1.3165
	424.	1.2494	415.	1.2527	406.	1.2622
23	424.	1.2496	415.	1.2648	406.	1.2922
	424.	1.2316	415.	1.2340	406.	1.2424
	424.	1.2315	415.	1.2249	406.	1.2222

(Continued)

(Sheet 2 of 3)

Table 16 (Concluded)

Pump Combination	Tank Level	Unit Cost	Tank Level	Unit Cost	Tank Level	Unit Cost
24	424.	1.2399	415.	1.2459	406.	1.2594
	424.	1.2373	415.	1.2333	406.	1.2341
	424.	1.2529	415.	1.2430	406.	1.2362
25	424.	1.2540	415.	1.2557	406.	1.2632
	424.	1.2617	415.	1.2563	406.	1.2543
	424.	1.2842	415.	1.2742	406.	1.2668
26	424.	1.2785	415.	1.2785	406.	1.2831
	424.	1.2902	415.	1.2849	406.	1.2824
	424.	1.3141	415.	1.3048	406.	1.2978
27	424.	1.3039	415.	1.3033	406.	1.3069
	424.	1.3167	415.	1.3119	406.	1.3097
	424.	1.3395	415.	1.3311	406.	1.3248

(Sheet 3 of 3)

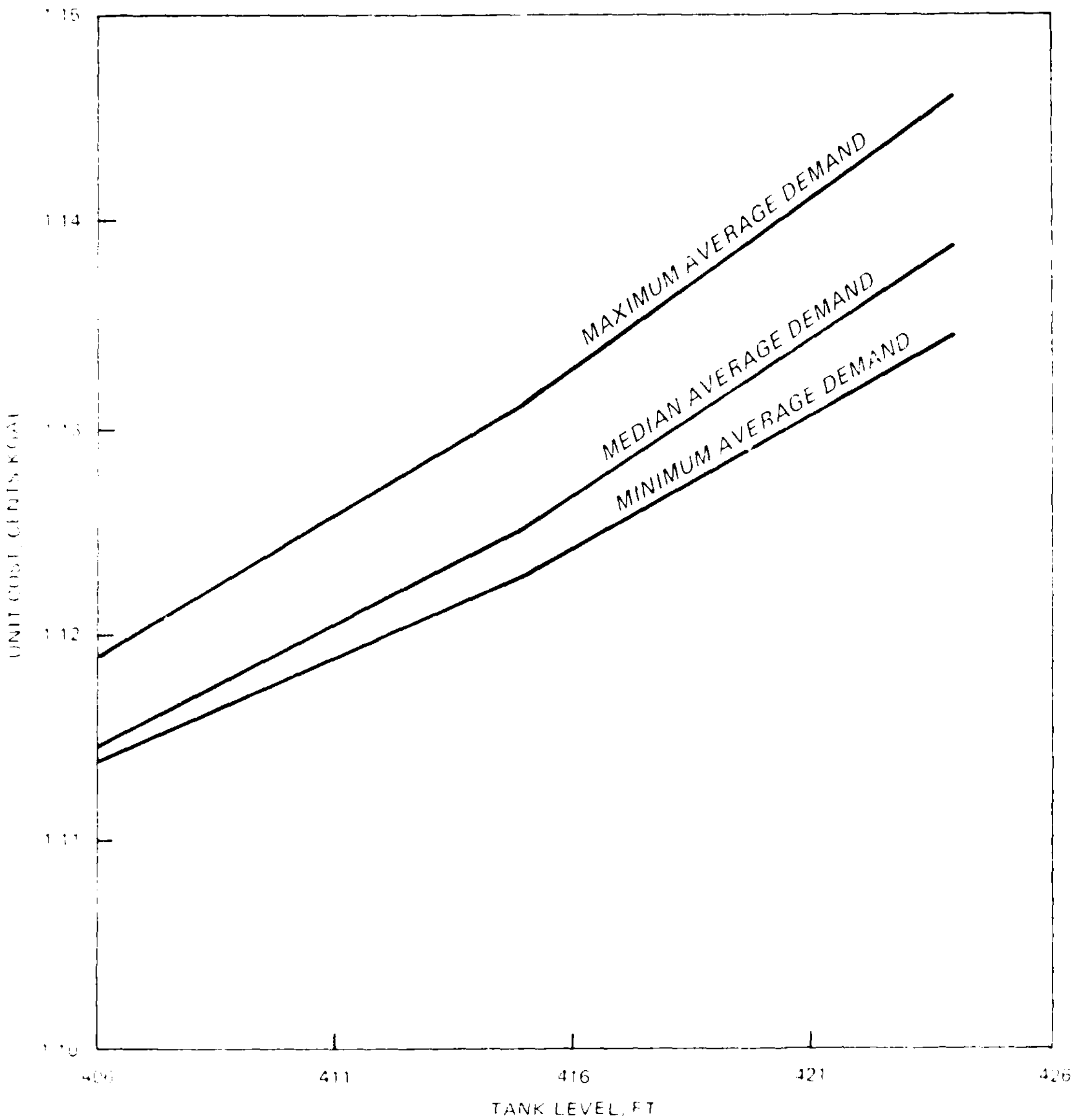


Figure 36. Typical set of TLC curves for third high-pressure system

$$Q_p = 27{,}980 + 0.2199*Q_d - 0.000000484*Q_d\hat{\ }2 \qquad \text{(2nd high)}$$

$$Q_p = 33{,}092 + 0.0374*Q_d - 0.000000175*Q_d\hat{\ }2 \qquad \text{(3rd high)}$$

where Q_d is the given system demand. For the typical set of TLC curves shown in Figures 35 and 36 and tank water levels of 330 and 419 ft, the following unit cost equations are obtained:

$$UC = 0.8175 - 2.00E\text{-}6*Q_d + 1.12E\text{-}12*Q_d\hat{\ }2 \qquad \text{(2nd high)}$$

$$UC = 1.4111 - 3.86E\text{-}7*Q_d + 2.72E\text{-}12*Q_d\hat{\ }2 \qquad \text{(3rd high)}$$

In each case, the quadratic equations were fitted using Lagrangian polynomials. For the equation form $Y = a + bX + cX\hat{\ }2$ the coefficients were determined using the following equations:

$$a = R0*X1*X2 + R1*X0*X2 + R2*X0*X1$$

$$b = [-R0*(X1+X2)] - [R1*(X0+X2)] - [R2*(X0+X1)]$$

$$c = R0 + R1 + R2$$

where

$$R0 = \frac{Y0}{[(X0-X1)*(X0-X2)]}$$

$$R1 = \frac{Y1}{[(X1-X0)*(X1-X2)]}$$

$$[illegible] = \frac{[illegible]}{[illegible]*(X2-X1)]}$$

[illegible] are three values of the independent variables (tank water [illegible] demand [illegible], and Y0, Y1, Y2 are the three correspond[illegible] pump flow [illegible] or unit cost UC).

[illegible] curves provide a simplified way for approximating [illegible] operating costs of the second and third high-pressure sys[illegible] demand and a desired tank level change, these [illegible] to determine the least costly pump combination [illegible] conditions. Alternatively, these curves can be [illegible] cost operation curves for use in generating [illegible] pressure zone.

Cost Operation Curves

74. Once the pump operation (TLF and TLC) curves were developed, a set of cost operation curves was generated. The cost operation curves are needed as input in the TOP and are used in determining an optimal tank trajectory for a given set of operating conditions. For both the second and third high-pressure systems, the rate of change in tank level was found to affect the hydraulics of the system much more than the actual tank level. As a result, the cost operation curves were developed as a function of tank filling (or draining) rate. For each pressure zone, three different curves were obtained (see Figures 37 and 38). The top curve in each case corresponds to the minimum cost associated with the situation where the tank is filling at a rate of 1 ft per hour. The middle curve corresponds to the minimum cost associated with the situation where the tank is neither draining nor filling. Finally, the bottom curve corresponds to the situation where the tank is draining at a rate of 1 ft per hour.

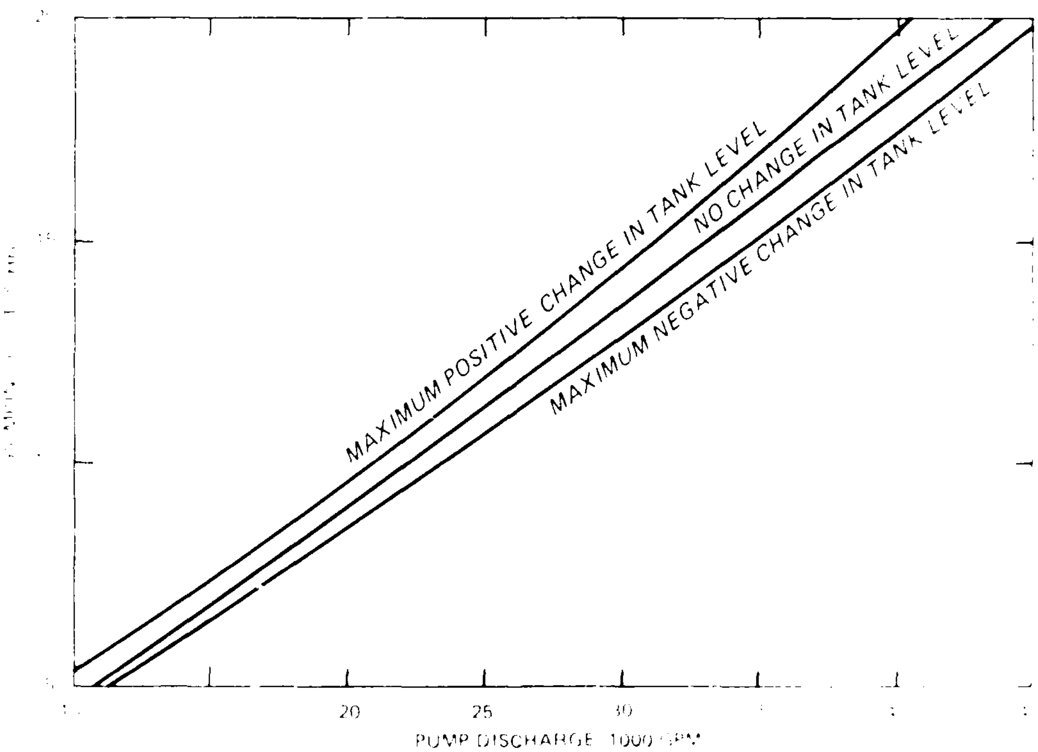
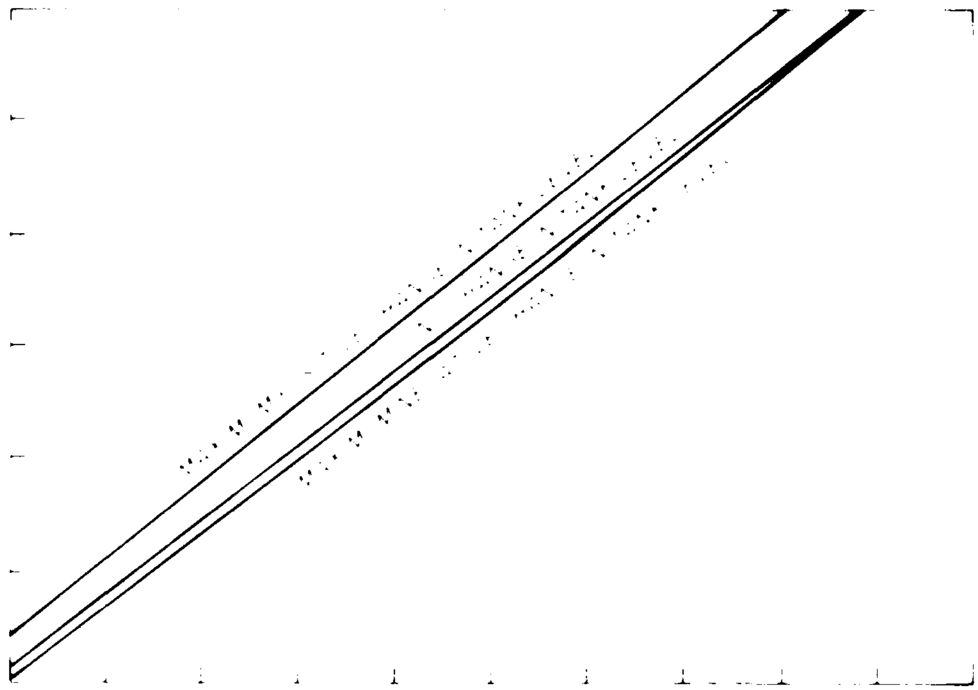


Figure 37. Cost operation curves for second high-pressure system



[illegible] high-pressure system

[illegible] the [illegible] typical day [illegible] pressure [illegible] time intervals. The optimal tank trajectories for the [illegible] pressure system are shown in Figures [illegible]. Comparison between the total costs associated with the optimal tank trajectories (obtained [illegible]) and the actual pump operation [illegible] is shown in Table [illegible]. The optimal tank trajectories for the high pressure system are shown in Figures [illegible]. Comparison between the total costs associated with the optimal tank trajectories (obtained [illegible]) and the actual pump operation [illegible] for each day is shown in Table [illegible].

Two of the days used in the study [illegible] had a variable electric rate schedule, while the other two days [illegible] had a constant rate schedule. In general, the days with a variable rate schedule had optimal policies, with the tank filling during periods of low electric rate and draining during periods of a peak rate. The optimal policies associated with the days with a constant electric rate schedule favored a more constant pumping rate. In both cases, however, significant energy savings were achieved. For those days with a variable electric rate, energy savings were obtained by pumping most of the water during periods when the electric rate was at a lower level. For those days with a constant electric rate, energy savings were obtained by maintaining a pump head that produced the maximum pump efficiency.

From an operational standpoint, it would be desirable to minimize the number of changes in pump operation during a day. Since the electric rate [illegible] changes in increments of [illegible] for weekdays, [illegible] time [illegible] represent the maximum practical operational period. To investigate the [illegible] of using a [illegible] operational period as opposed to a shorter period [illegible] optimal trajectories for both zones for all [illegible] days were obtained [illegible] time intervals. For each day, the resulting trajectories [illegible] were [illegible]. As a result, an operational time interval of [illegible] recommended.

[illegible] an estimate of the annual savings [illegible] associated with each typical day were [illegible] days in the year. The number of [illegible] can be summarized as follows: winter weekends [illegible] days, winter weekdays [illegible] days, summer weekends [illegible] summer weekdays [illegible] days. Based on these values, the total annual

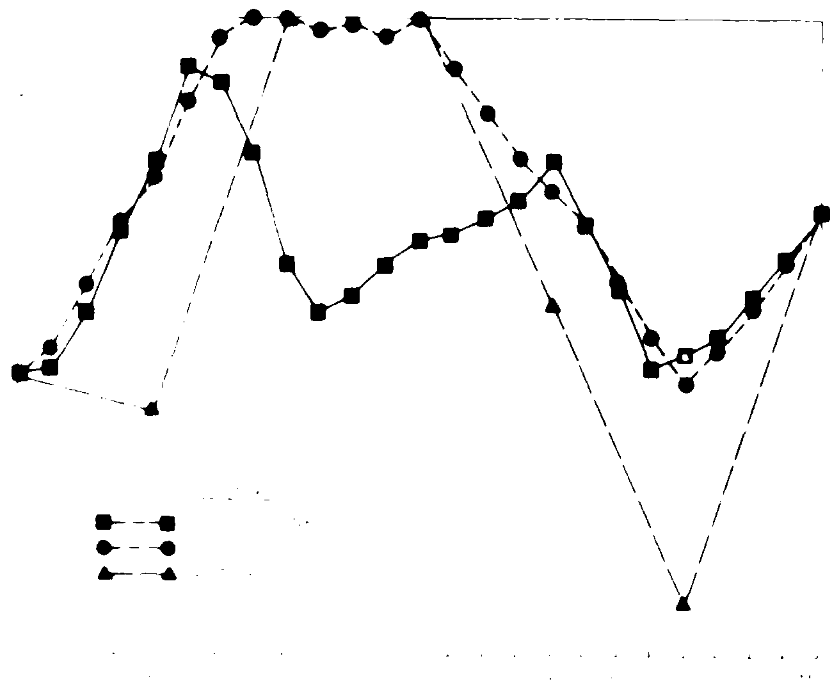
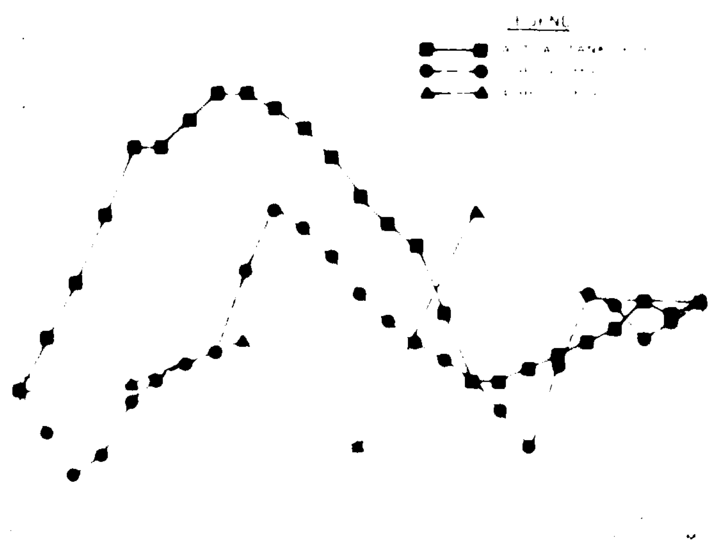


Figure ... ptimal tank trajectories for second high-pressure system, Thursday, 20 March 1986



... ttima tank trajectories f ... ond high pressure system, Saturday, ... Mar ...

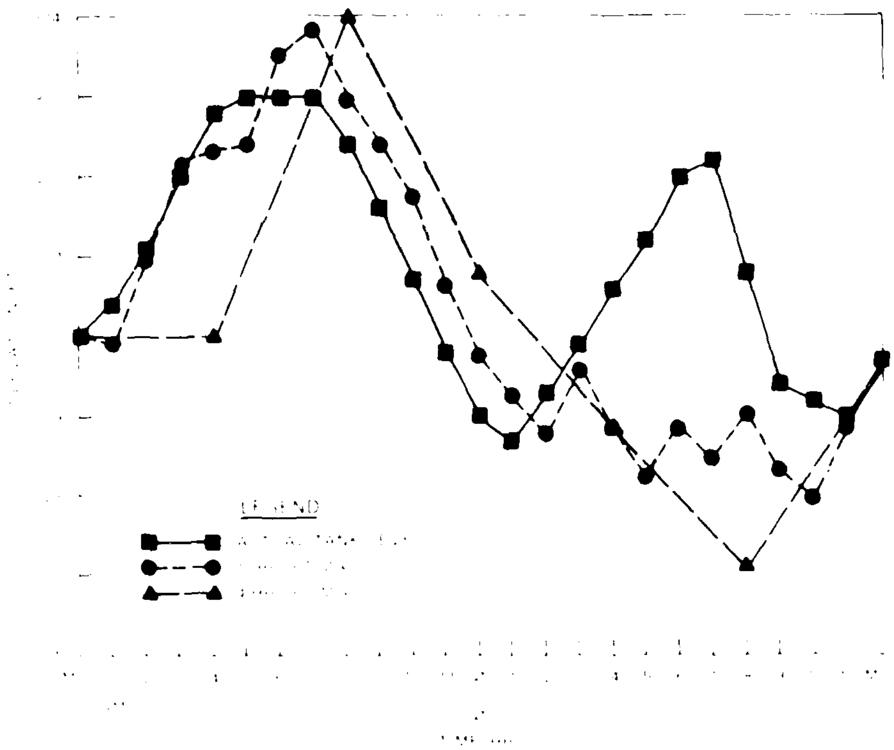


Figure 41. Optimal tank trajectories for second high-pressure system, Sunday, 8 June 1986

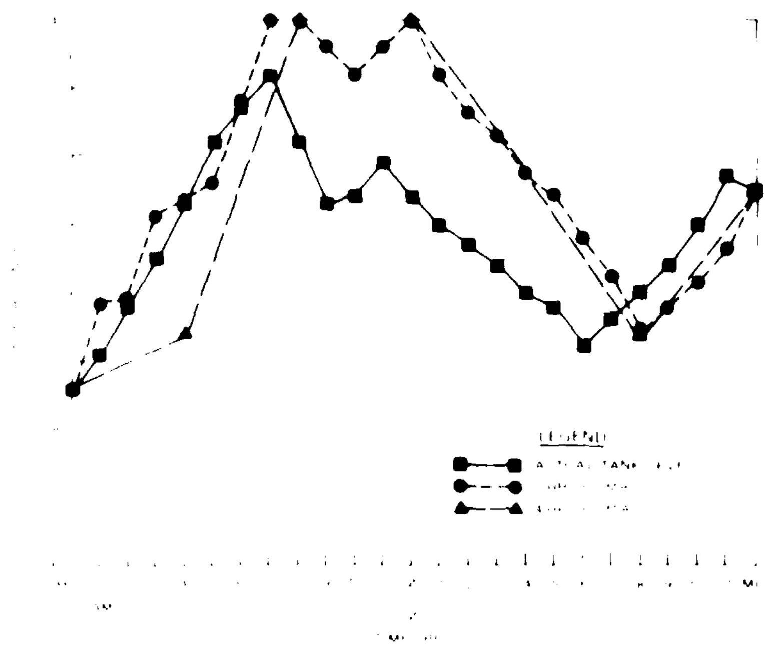


Figure 42. Optimal tank trajectories for second high-pressure system, Wednesday, 11 June 1986

Table 1[illegible]

Energy Costs for Second High-Pressure System

Date, 1986	Type of Day	Actual Policy $	1-hr Optimal Policy $	Percent Differ- ence	[illegible]-hr Optimal Policy $	Percent Differ- ence
20 March	Winter weekday	1,371	1,310	[illegible]	[illegible]	[illegible]
[illegible] March	Winter/weekend	981	[illegible]	[illegible]	[illegible]	[illegible]
8 June	Summer/weekend	1,231	1,1[illegible]	[illegible]	[illegible]	[illegible]
11 June	Summer/weekday	1,983	[illegible]	[illegible]	[illegible]	[illegible]
Projected annual cost		517,615	[illegible]	[illegible]	[illegible]	[illegible]
Projected annual savings			28,[illegible]		[illegible]	

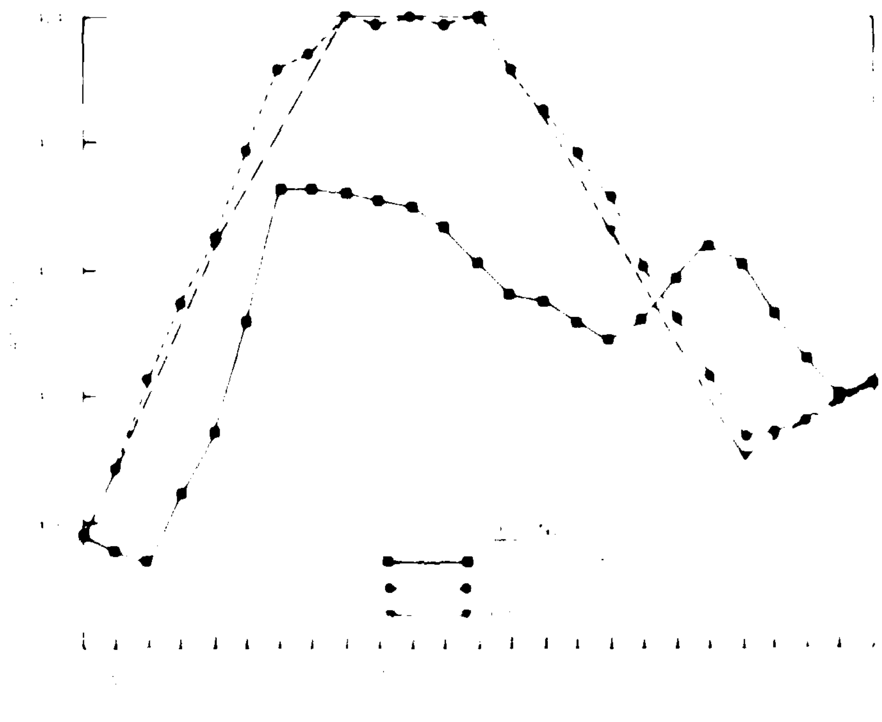
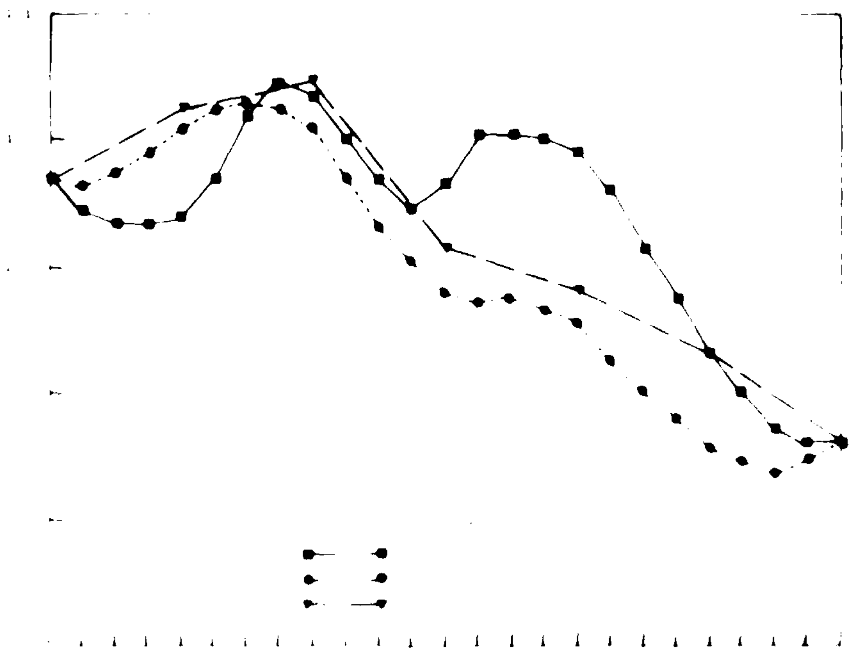
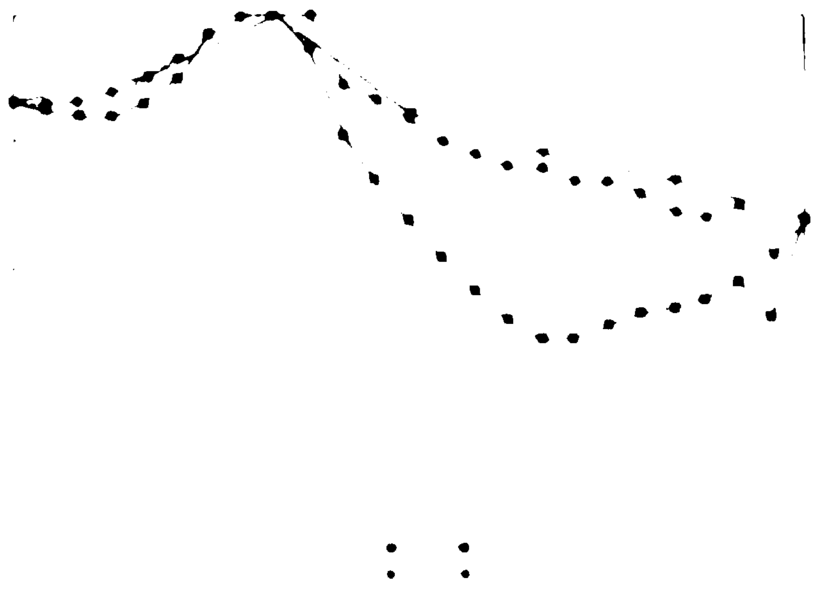


Figure [illegible] [illegible]ptimal [illegible] trajectories [illegible] system, Thursday, [illegible]





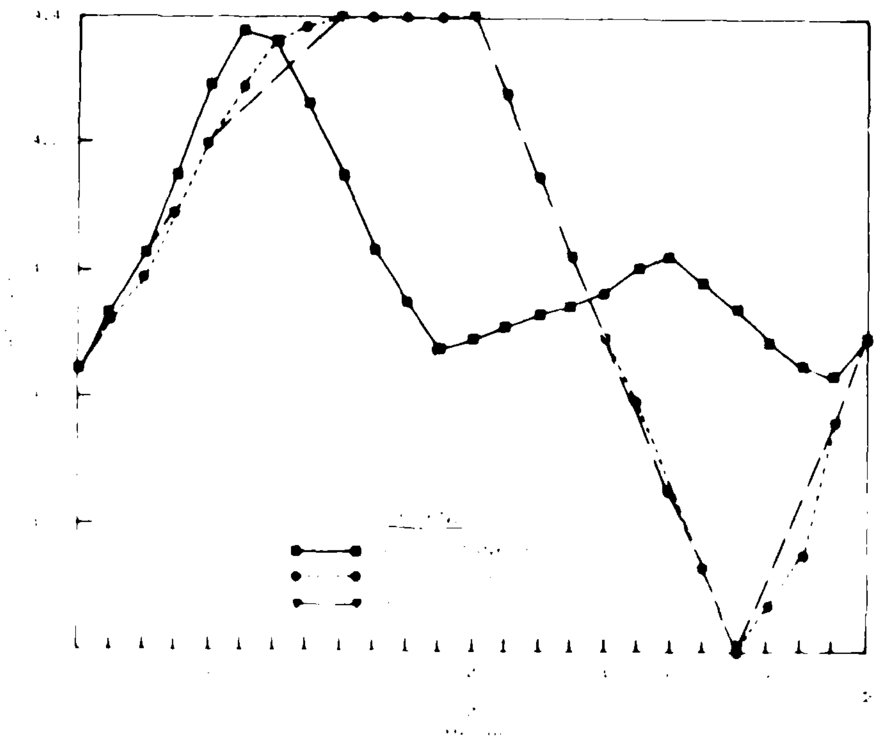


Figure [illegible]. [illegible]timal tank trajectories for third high-pressure system, Wednesday, [illegible] June 1986

Table [illegible]

[illegible] high pressure system

energy usage cost savings for the second high-pressure system were estimated to be $25,330 (5 percent), while the total annual energy usage cost savings for the third high-pressure system were estimated to be $64,583 (7.5 percent). Similar percentage savings can be expected for both the low-service and first high-pressure zones.

Optimal Pump Operation Policies

80. After the optimal tank trajectories for each day for each system were determined, the corresponding pump operation schedules were determined by reapplying the PCP for each time period in the tank trajectory. Given discharge, tank fill rates, and tank levels, the PCP was used to determine which pumps should be operated. The optimal pump operation schedules obtained using the 1-hr time interval are shown in Figures 47-54. The cost in dollars associated with each hour of operation is shown along the bottom of each figure. For each pump, the period of operation is indicated by the shaded area. Since the characteristic curves for the pumps associated with each pumping station were essentially the same, the important information obtained from each figure is the number of pumps (for each pumping station) operating at a particular time as opposed to the actual pumpage.

81. In order to evaluate the sensitivity of the resulting pump operation policies to the approximations involved in the pump operation and cost generation curves, each day was simulated using the resulting optimal pump policies and the calibrated hydraulic network model. The tank level trajectories resulting from these simulations were then superimposed on the results of the TOP runs, as shown in Figures 55-62.

82. As can be seen from these figures, the desired optimal tank trajectories obtained from the TOP were essentially matched through the implementation of the pump operation policy (obtained from the PCP). The high level of correlation between the two sets of curves illustrates the accuracy of the pump operation curves in representing the hydraulics of the modeled systems.

Sensitivity Analysis

83. In addition to generating optimal pump operating policies for com- [illegible], several additional case studies were

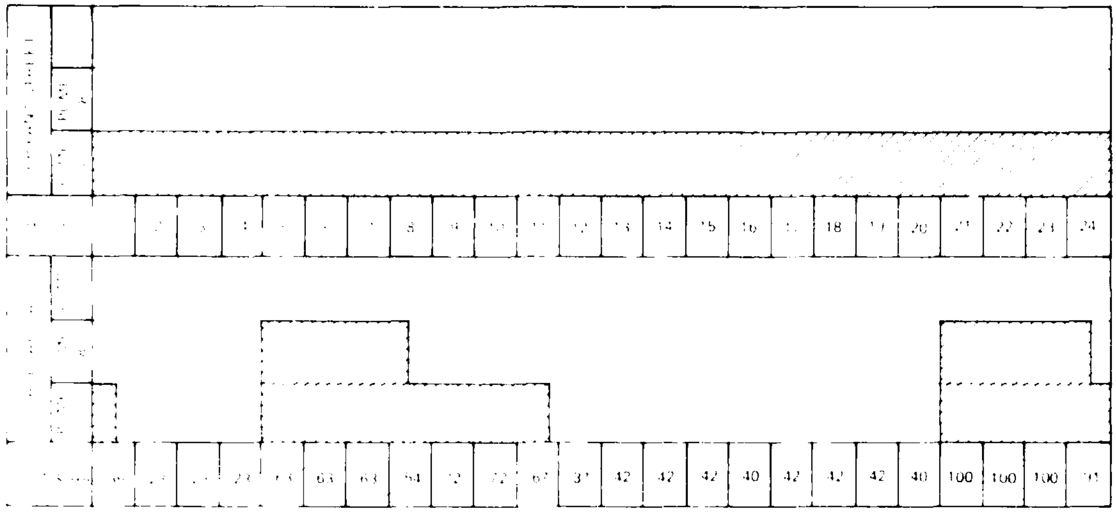


Figure 47. Optimal pump operation schedule for second high-pressure system, Thursday, 20 March 1986

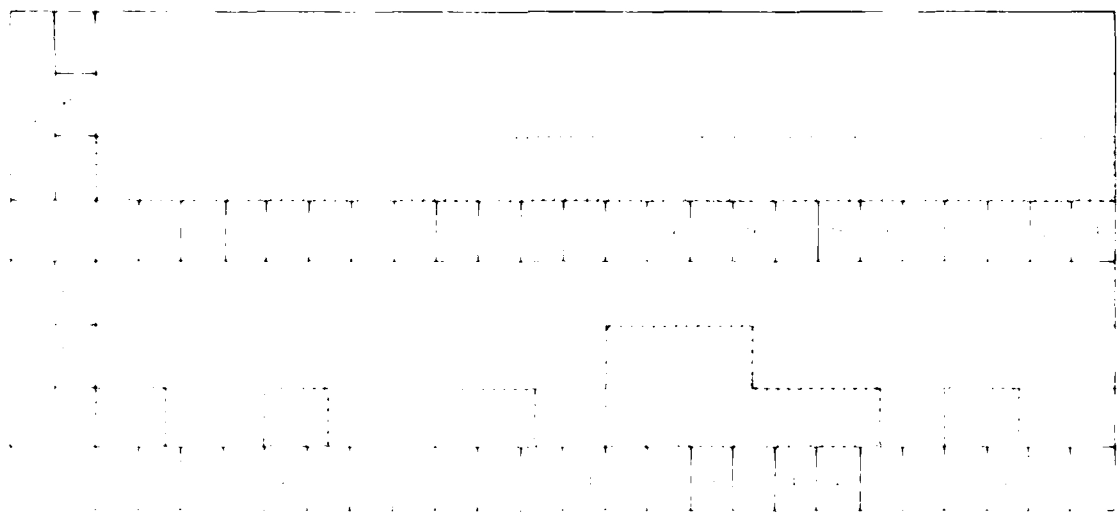


Figure [illegible]. Optimal pump operation schedule for second high-pressure system Saturday, [illegible] March 1986

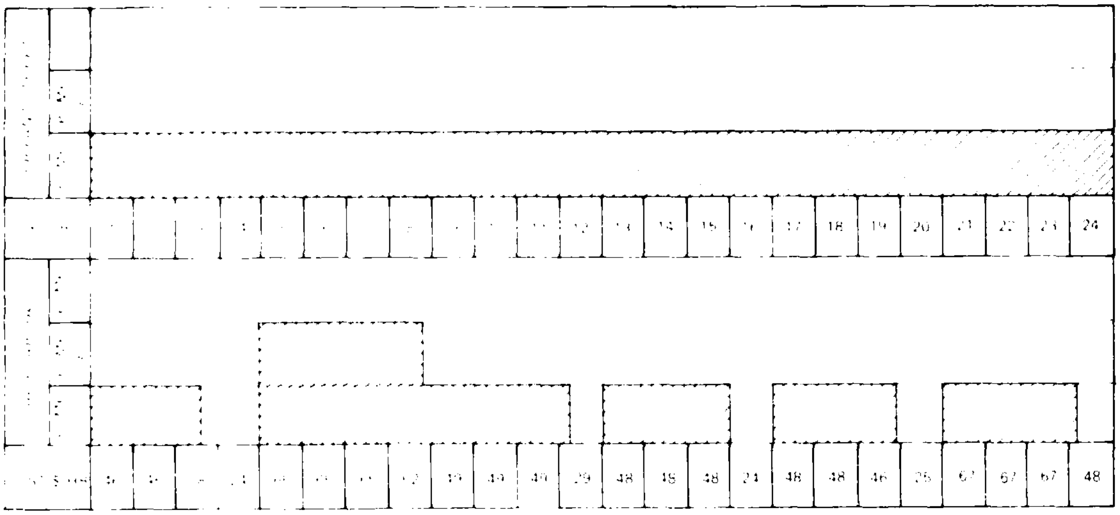


Figure 49. Optimal pump operation schedule for second high-pressure system, Sunday, 8 June 1986

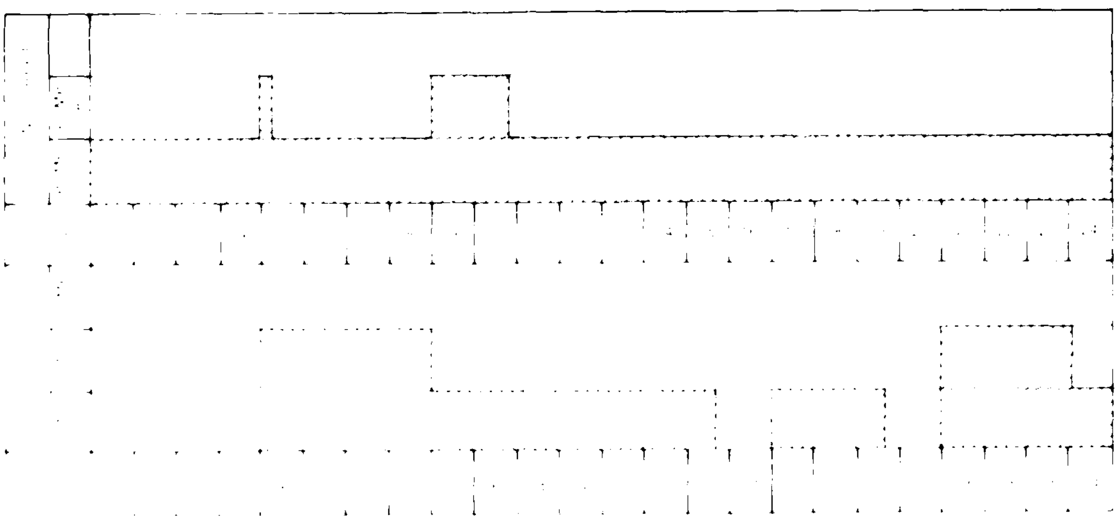


Figure . [illegible]ptimal pump operation schedule for second high-pressure system, Wednesday, 11 June 1986

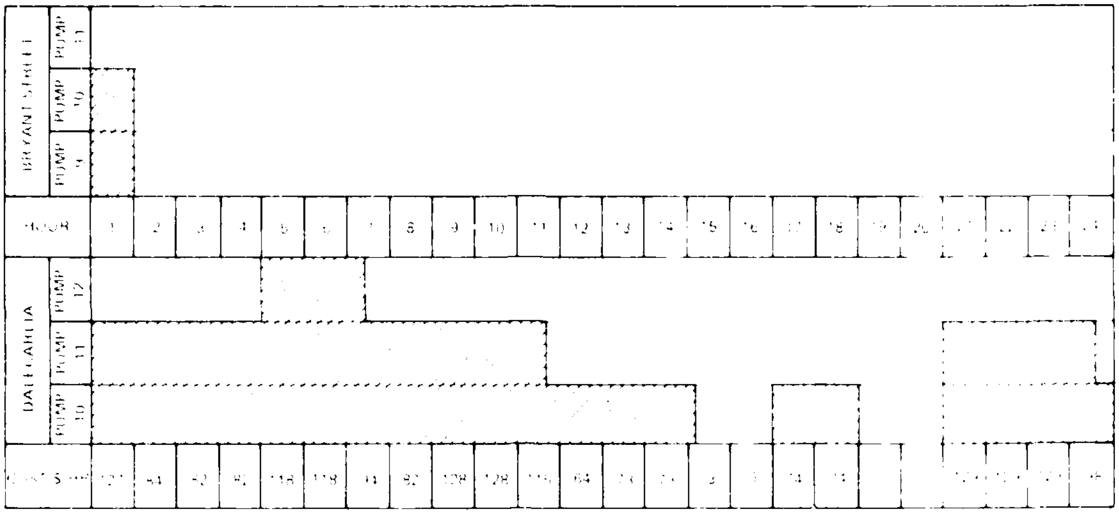


Figure 51. Optimal pump operation schedule for third high-pressure system, Thursday, 20 March 1986

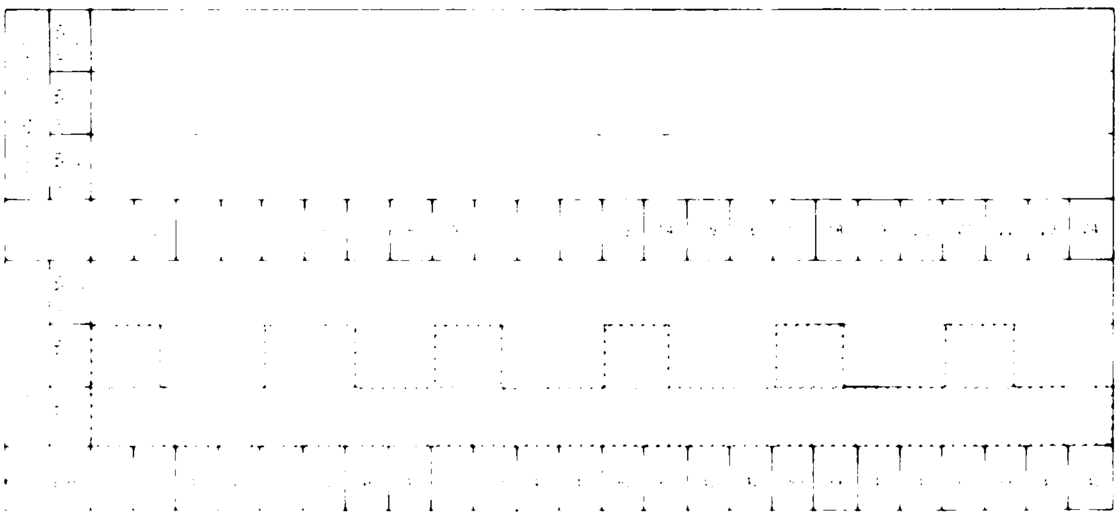


Figure 52. Optimal pump operation schedule for third high-pressure system, Saturday, 29 March 1986

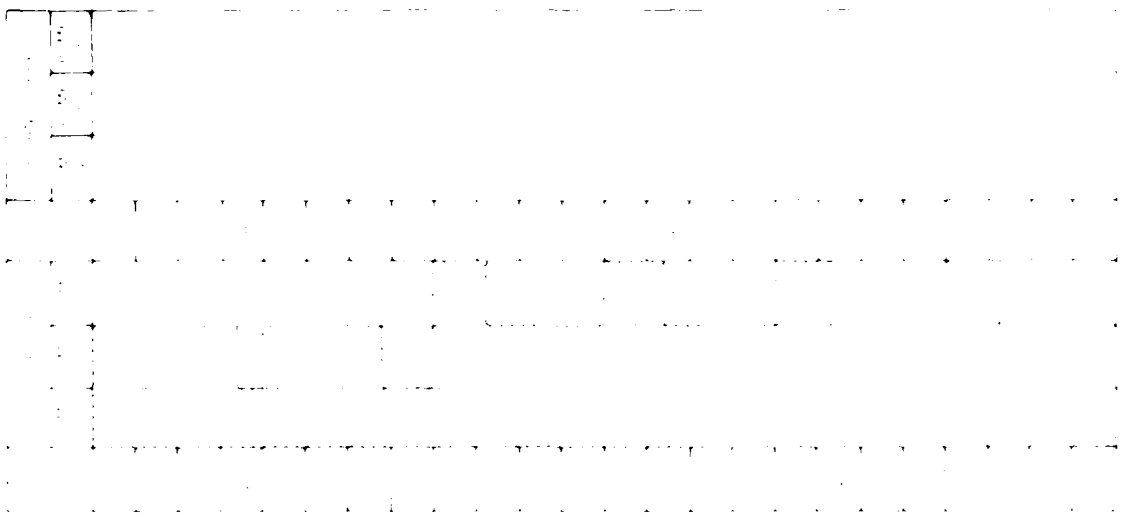
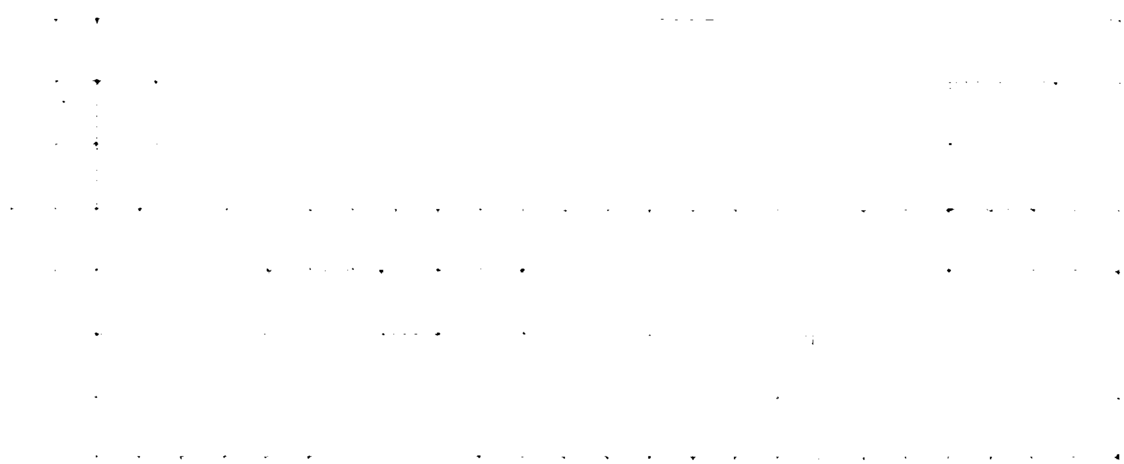
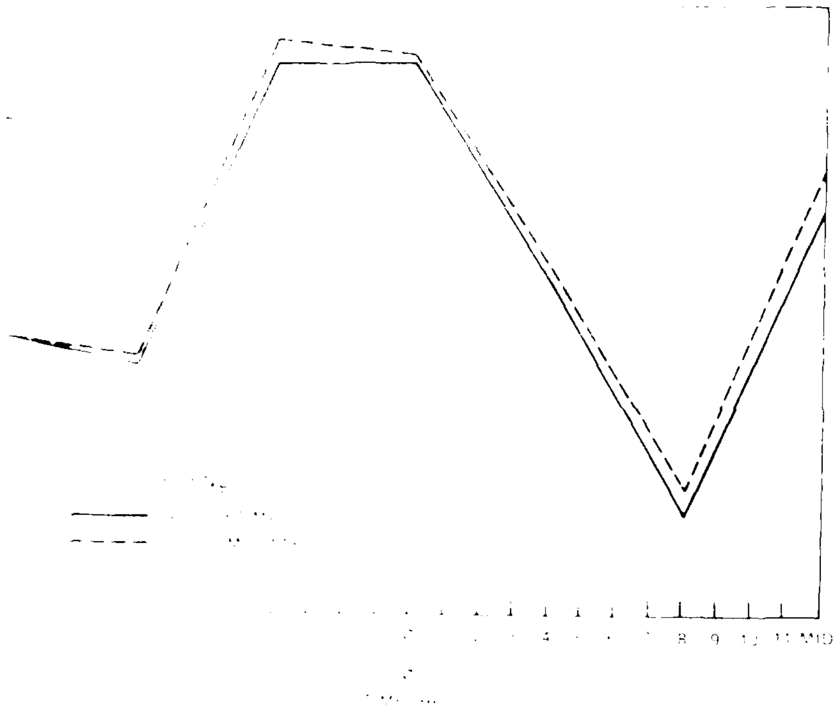
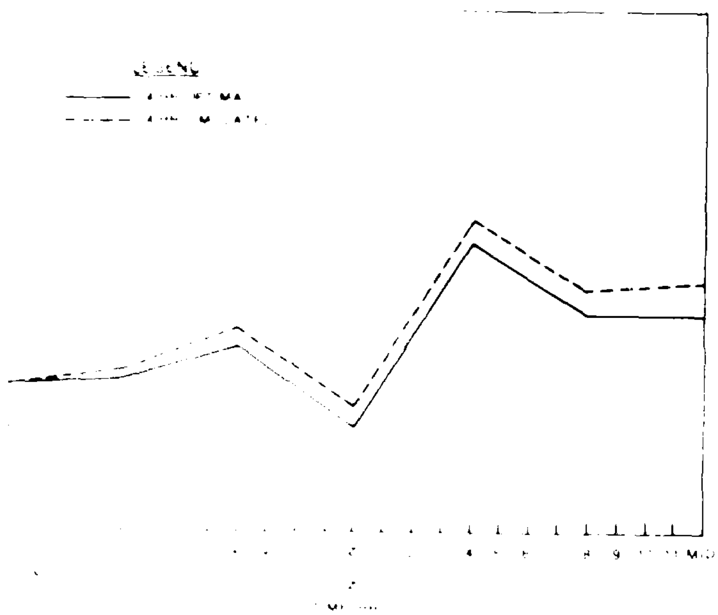


Figure [illegible]. Optimal pump operation schedule for third high-pressure system, Sunday, [illegible] June [illegible]





[illegible] imulation results for second high-pressure system, Thursday, 20 March 1986



[illegible] imulation results for second high-pressure system, Saturday, 29 March 1986

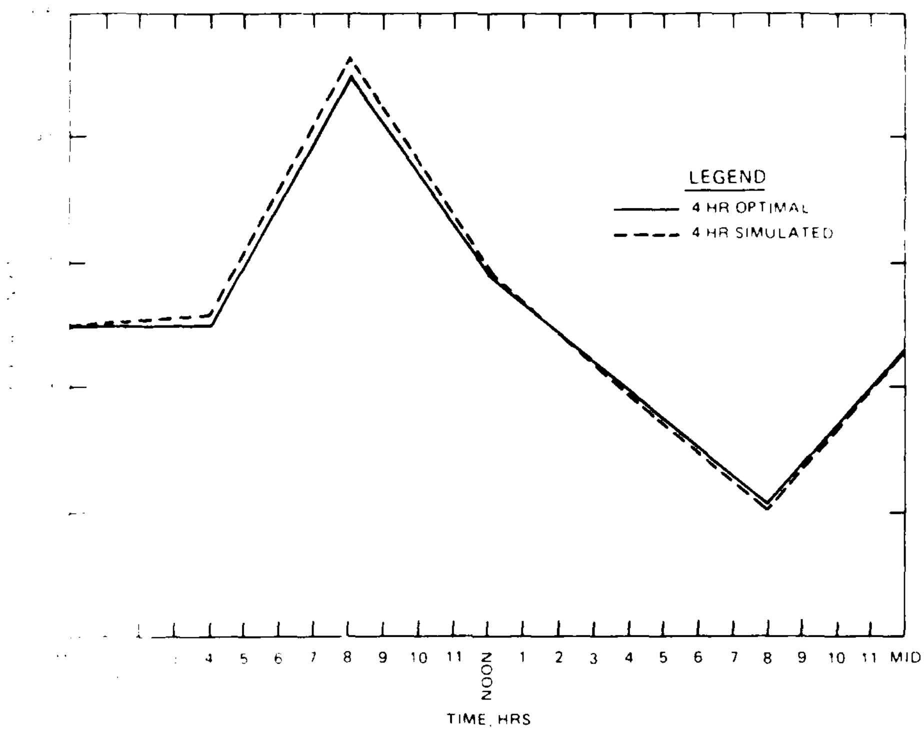
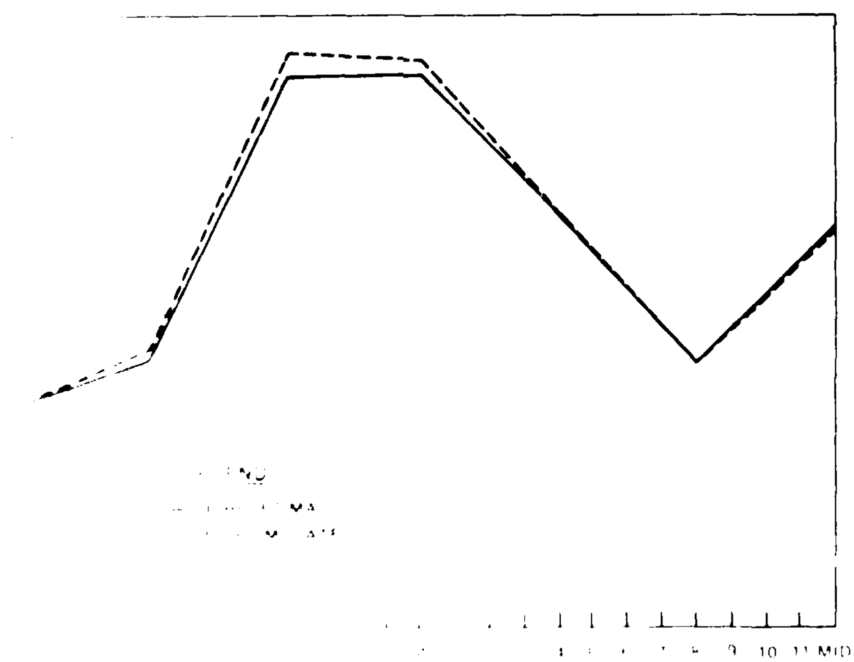


Figure 57. Simulation results for second high-pressure system, Sunday, 8 June 1986



second high-pressure

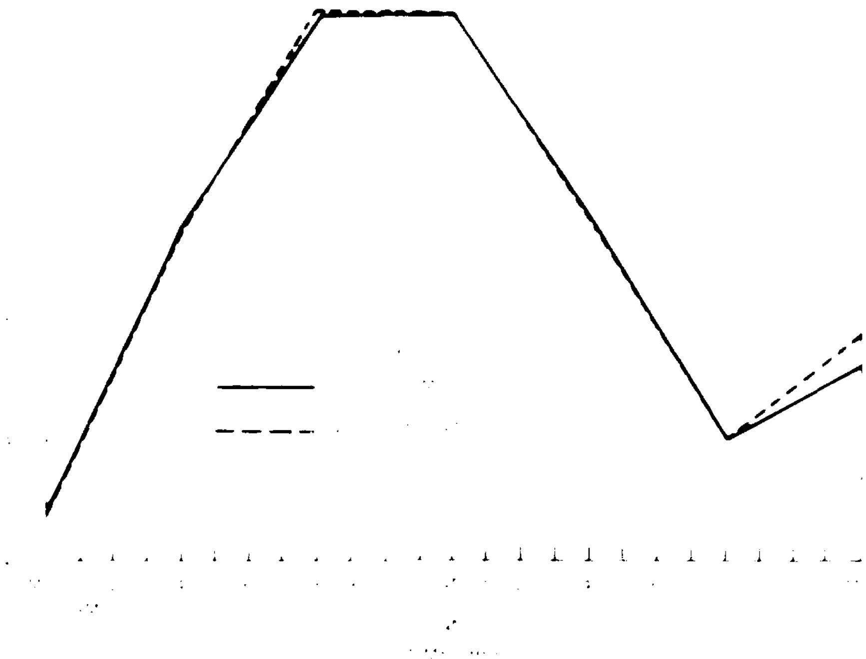


Figure 59. Simulation results for third high-pressure system, Thursday, 20 March 1986

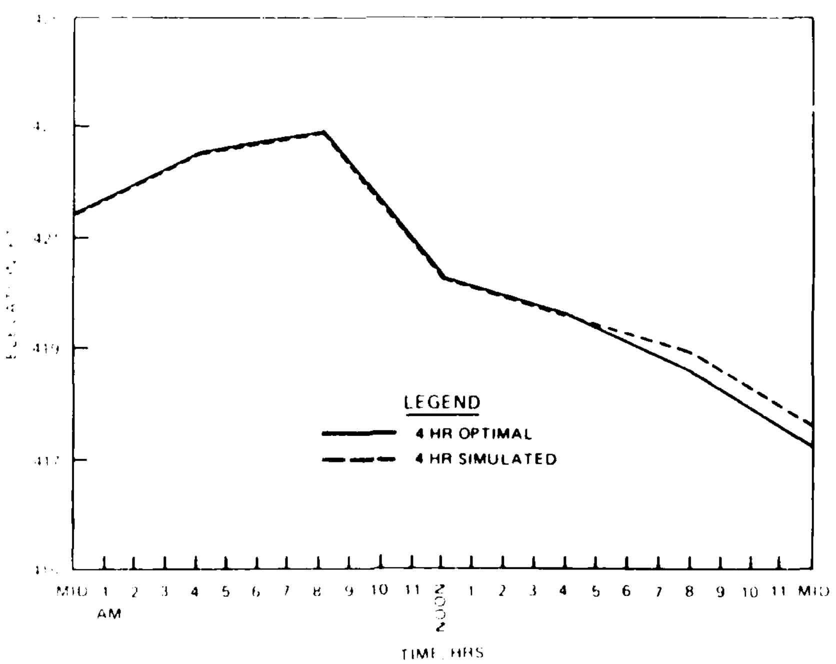


Figure 60. Simulation results for third high-pressure system, Saturday, 29 March 1986

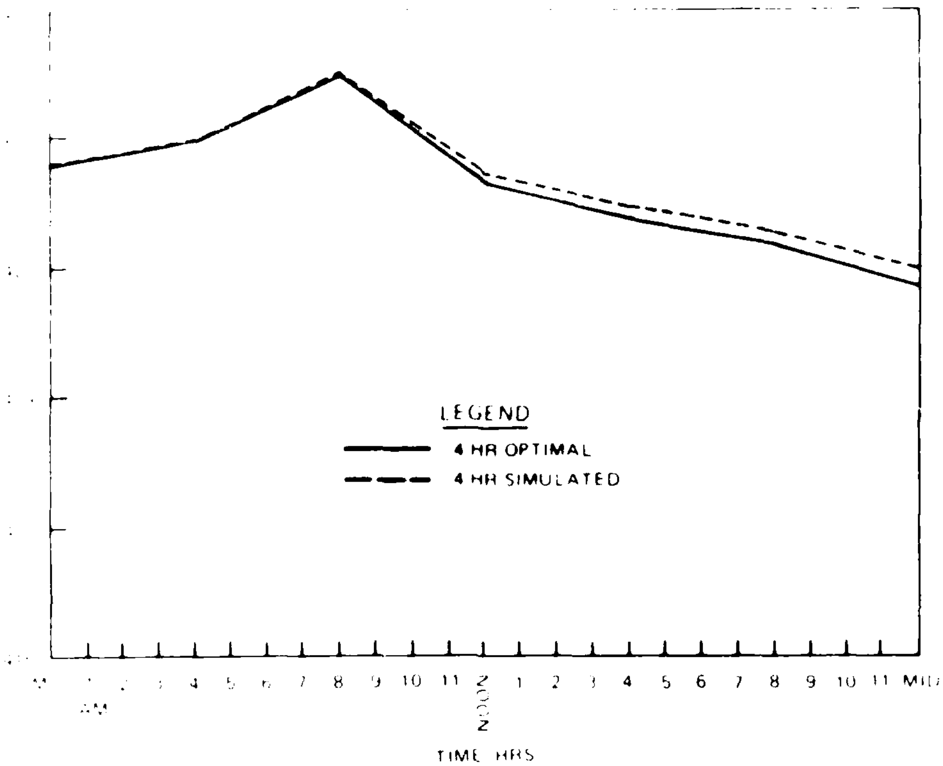


Figure 61. Simulation results for third high-pressure system, Sunday, 8 June 1986

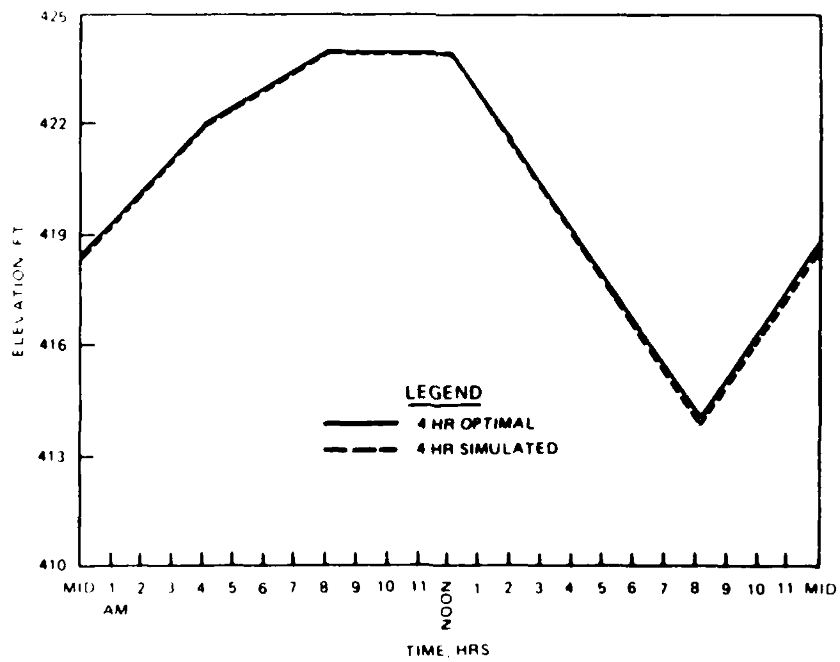


Figure 62. Simulation results for third high-pressure system, Wednesday, 11 June 1986

[illegible] to determine the effect of different operating conditions on the resulting policies. Four case studies were examined: (a) alternative initial and final tank water levels, (b) additional storage, (c) weekly operating policies, and (d) response to extreme demand events such as a fire or a water-line break.

Alternative initial and
final tank water levels

[illegible]. For each optimal tank trajectory, the initial and final tank water levels were fixed at the levels obtained from the actual operating data. This was done to ensure a common basis for comparing the results of the optimal solutions with the actual operating policy. It is quite possible that the optimal solutions could be improved by allowing the initial and final states to vary. To investigate this possibility, each day was reanalyzed by requiring the initial and final tank water levels to be set at full and half-full starting at midnight. The results of this analysis are summarized in Tables [illegible] and [illegible].

85. A direct comparison between the original optimal solutions and the solutions obtained from the modified starting levels is not possible because the original solutions did not stop and start at the same elevation (see Figures 39-46). What is possible is a direct comparison between the solutions for the cases when the tanks started and ended at half-full and the cases when the tanks started and ended at full. From this comparison it would appear that additional savings may be obtained by having the tanks at half-full at midnight as opposed to full. For weekends, the starting elevations do not appear to affect the overall cost.

Additional storage

86. The second case involved an examination of the impact of providing additional storage. For this study the tank volumes for both systems were doubled. This volume was considered to be a realistic estimate of potential additional storage. Optimal tank trajectories for each day for both pressure zones were then obtained by applying TOP to the modified systems. The total costs associated with the resulting trajectories are presented in Tables 21 and 22. From these results it would appear that addition of more storage to the second high-pressure system would result in an increase in energy usage costs. (The increase was due to initial tank water levels rather than a

Table 19

Optimal Cost for Second High-Pressure System with Alternate Initial and Final Water Levels

Date, 1986	Type of Day	4-hr Optimal $	Tank Full $	Tank Half-Full $
20 March	Winter/weekday	1,327	1,316	1,262
29 March	Winter/weekend	953	924	924
8 June	Summer/weekend	1,170	1,179	1,178
11 June	Summer/weekday	1,825	1,824	1,729
Projected annual cost		492,285	488,659	471,529
Projected annual savings			3,626	20,756

Table 20

Optimal Cost for Third High-Pressure System with Alternate Initial and Final Water Levels

Date, 1986	Type of Day	4-hr Optimal $	Tank Full $	Tank Half-Full $
20 March	Winter/weekday	2,002	2,046	1,906
29 March	Winter/weekend	1,410	1,577	1,574
8 June	Summer/weekend	2,090	2,169	2,168
11 June	Summer/weekday	3,274	3,427	3,255
Projected annual cost		797,114	833,082	794,817
Projected annual savings			-35,783	2,482

Table 22

Optimal Cost for Third High-Pressure System with Additional Storage

Date, 1986	Type of Day	4-hr Optimal $	Additional Storage, $	Savings $
20 March	Winter/weekday	2,002	1,989	13
29 March	Winter/weekend	1,410	1,245	165
8 June	Summer/weekend	2,090	2,102	7
11 June	Summer/weekday	3,274	3,075	199
Projected annual cost		797.144	765,932	
Projected annual savings				31,182

[illegible] optimal pump trajectories, the total amount of water being pumped is also shown. As can be seen from the figures, operation policy for the weekends is different from the policy for the weekdays. For the weekdays, each optimal trajectory consisted of filling the tank during the off- or intermediate-peak periods and draining the tank during the on-peak period. During the weekends the tanks were gradually filled.

Recovery from extreme demands

88. In the final case study, the impact of an extreme demand (i.e., a fire or main break) on the optimal weekly trajectories was examined. For each week a trajectory was begun at 8 pm on Monday night with the tanks empty. This was considered to be the extreme case for a fire or a main break. The objective of each run was to examine how long it took for the optimal trajectory to recover from an extreme event. The optimal costs and trajectories resulting from these runs are shown in Figures 67-70. As shown, the system can recover from such an event very rapidly. For each case, the optimal trajectory returns to a typical pattern within a day of the event.

Impact of Electric Demand Charges

89. The total electric charges for each pumping station are a function of three different charges: usage charges, production and transmission charges, and distribution charges. In the current study, only the usage charges have been considered. In developing an optimal pump operation policy for both the second and third high-pressure systems, it has been assumed that

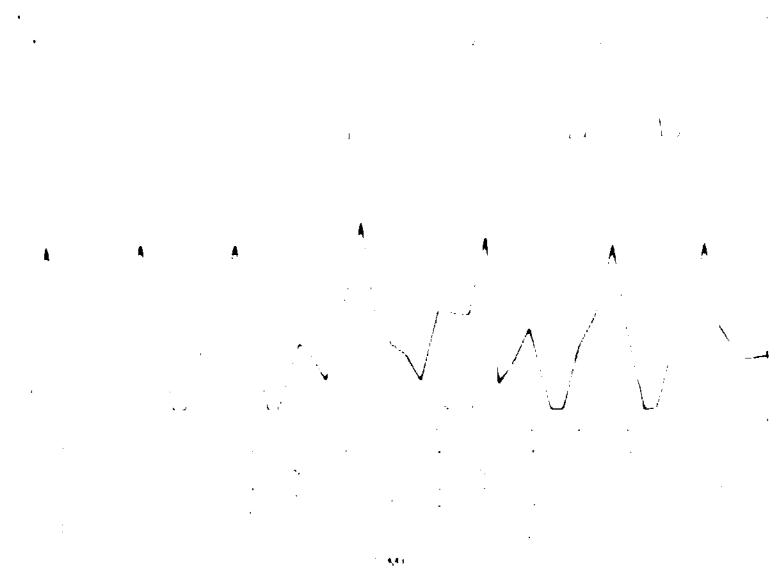


Figure [illegible]. Week-long simulation for second high-pressure system during winter months

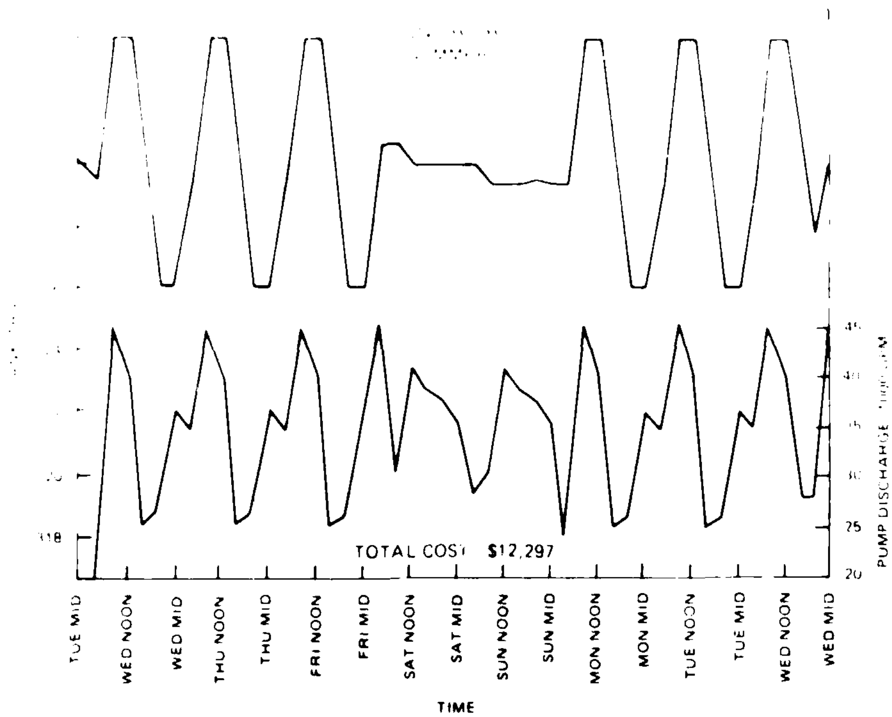


Figure 64. Week-long simulation for second high-pressure system during summer months

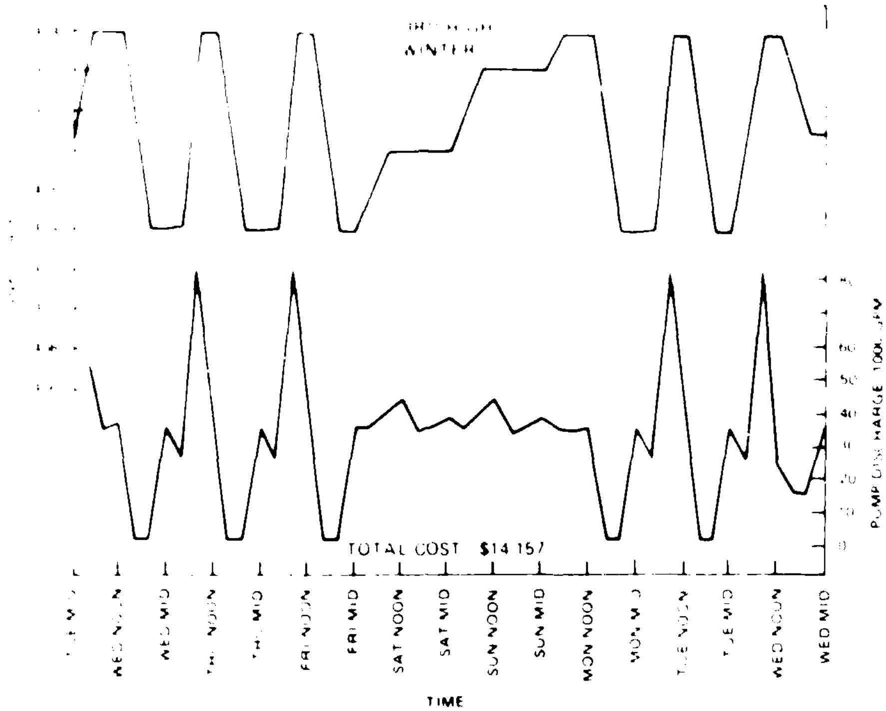


Figure 65. Week-long simulation for third high-pressure system during winter months

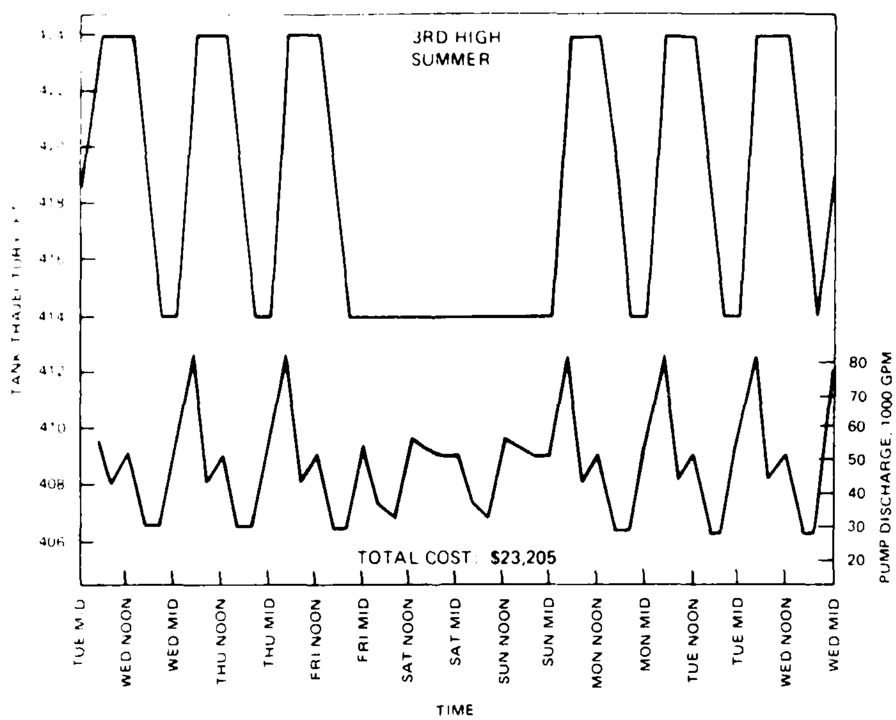


Figure 66. Week-long simulation for third high-pressure system during summer months

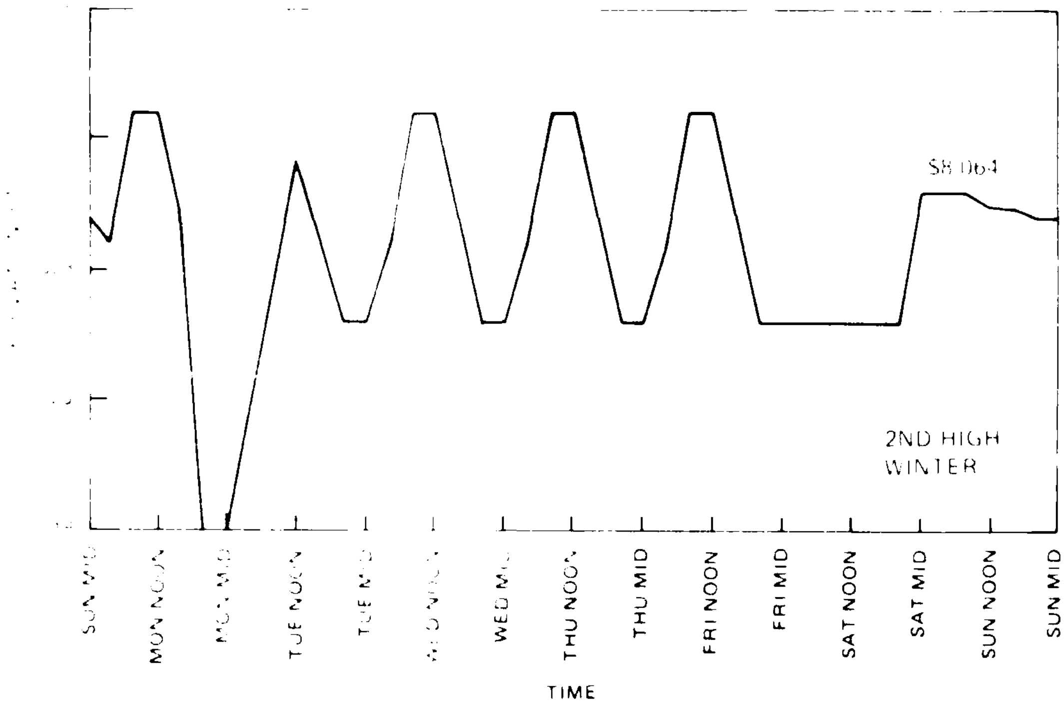


Figure 67. Optimal tank trajectory for recovery from emergency for second high-pressure system during winter months

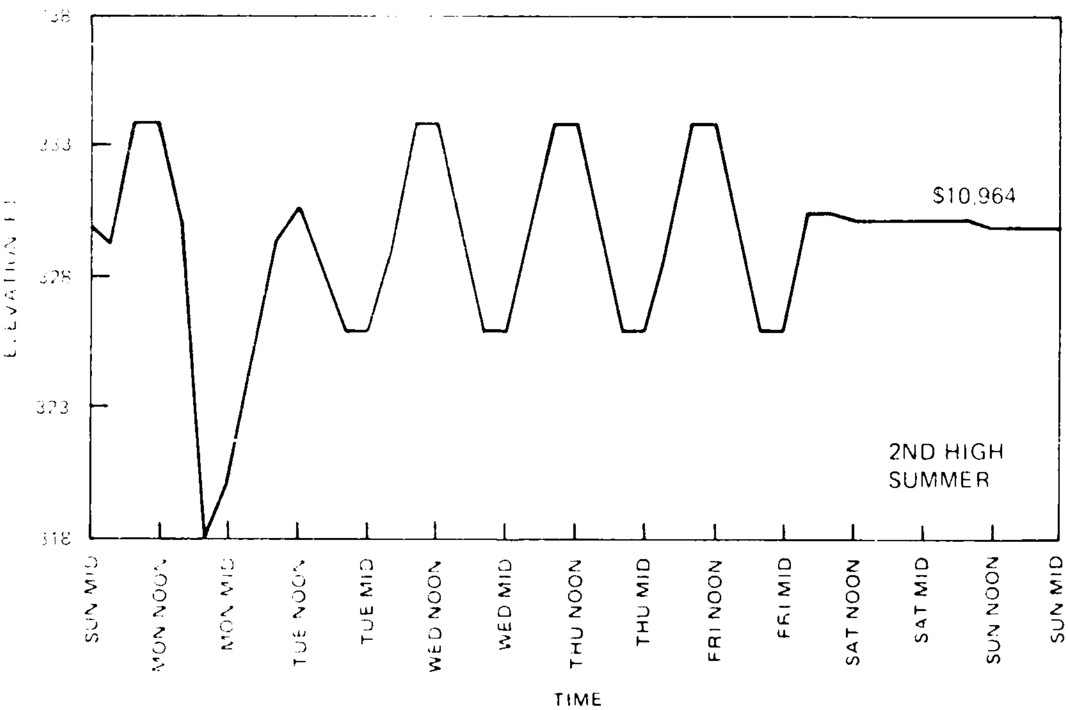


Figure 68. Optimal tank trajectory for recovery from emergency for second high-pressure system during summer months

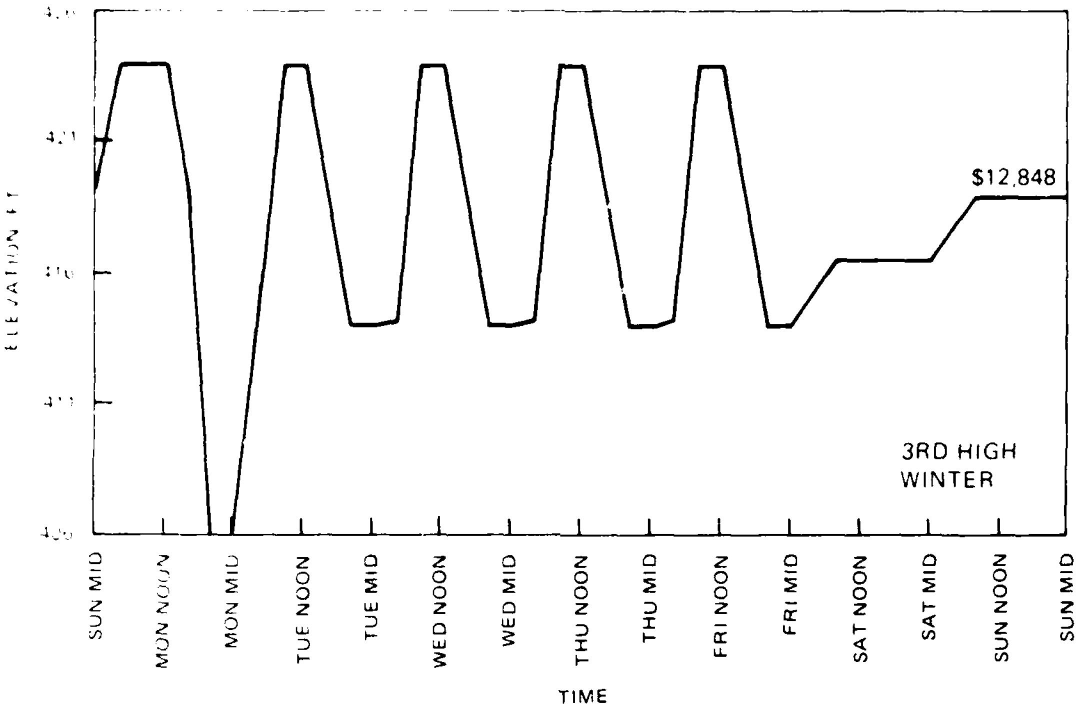


Figure 69. Optimal tank trajectory for recovery from emergency for third high-pressure system during winter months

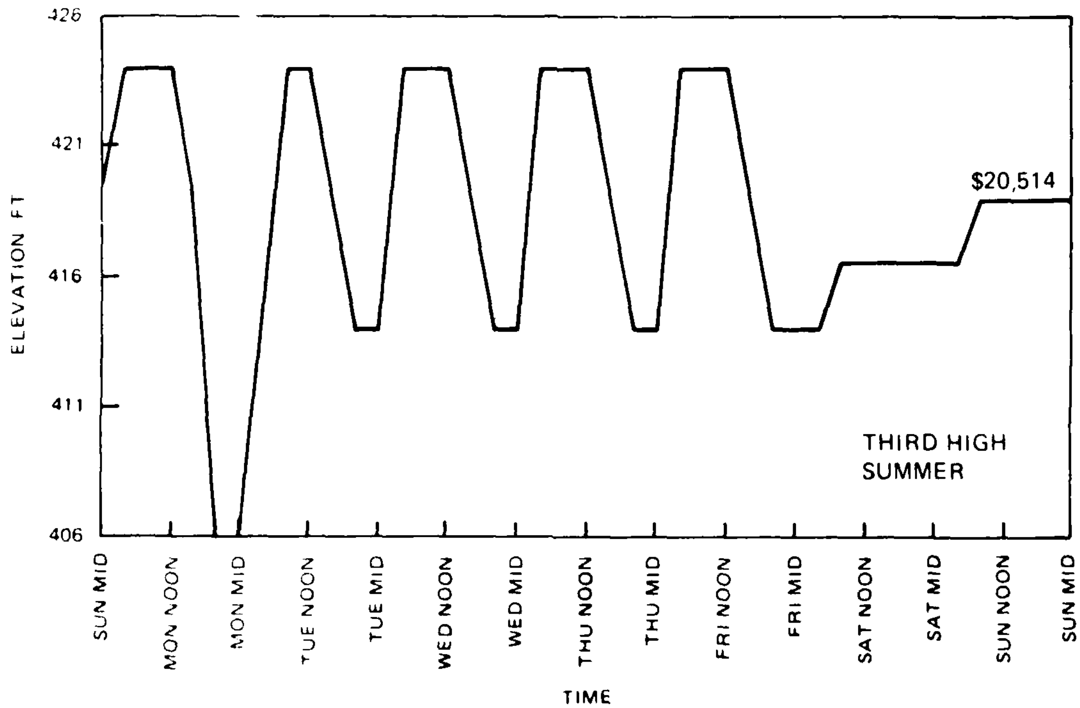


Figure 70. Optimal tank trajectory for recovery from emergency for third high-pressure system during summer months

both systems are essentially independent. Thus, the optimal operating policy for one pressure zone will not adversely affect the optimal operating policy for another zone. While this is a valid assumption when only energy usage charges are considered, some compromises may be required when production, transmission, and distribution charges are considered.

90. For both Dalecarlia and Bryant Street pumping stations, production and transmission charges are based on the maximum 30-min demand recorded during the on-peak weekday period (noon to 8 pm) for all summer months (June through September). This charge is based on the total electrical demand (in kilowatts) for each pumping station. If the operators are not careful in pump selection, it is possible that the additional energy cost savings resulting from the implementation of the optimal operating policy for each pressure zone could be offset by an increase in the production and transmission charges for the entire pumping station. For this study, however, the optimal solutions for days with variable rate schedules were characterized by pumping during periods of nonpeak electric rates. As a result, it is quite possible that the overall costs will be even further decreased by implementation of the optimal operating policy.

91. Similar to the production and transmission charges, the distribution charges for each pumping station are also based on a maximum 30-min demand. This demand charge, however, is not restricted to the on-peak period nor the summer months. Instead, the distribution charge for each month is based on the maximum 30-min demand recorded during the previous 12-month period. In general, this kind of charge can be minimized by using electricity at a constant rate. From an overall perspective this charge may be further reduced by dividing the load as equally as possible between the Dalecarlia and Bryant Street stations.

92. As with the production and transmission charges it is possible that the additional savings resulting from implementation of the optimal operating policy could be offset by an increase in the distribution charge. However, due to the nature of the yearly system demand schedule and the existing electric rate schedule, only 1 month of the year will usually control the monthly demand charge for that year. For the DC system this month is typically either August or September. For the controlling month, the demand charge will normally be more important than the electric usage charge because the resulting demand charge will be in effect for the next 11 months. During this month the

system should be operated with the objective of [illegible]. During the rest of the [illegible], the system may be operated [illegible] optimal tank trajectories as long as the peak demand is not exceeded. The maximum demand constraint may be imposed on the [illegible] either externally by the operator or internally [illegible] directly into the optimization program.

Extension to Multitank Systems

93. As mentioned earlier, application of the [illegible] procedures to the second and third high-pressure areas was [illegible] the systems behaved as if they were each controlled by a single tank, which meant that the state of the system could be completely described [illegible] tank level. For this analysis, 100 discrete states, which [illegible] discrete tank levels, were used. If there were two tanks that [illegible] the same range, it would be necessary to use [illegible] tanks would require 100^n states. This is referred to as the "curse of dimensionality" and can be a serious problem for dynamic programming. The large number of states would greatly increase the computation time, may lead to [illegible] overflow of computer memory, or may necessitate fewer states with an associated loss in accuracy.

94. The problem, however, is not quite as serious as it may seem. If multiple tanks are so far apart that they behave independently, only the storage tank near the pumping station need be considered in determining states. In most cases with multiple tanks, the tanks will neither be so close that they can be treated as one nor so far apart that one can be ignored. However, it is not necessary to increase the number of states as dramatically as described in the previous paragraph.

95. The key to limiting the number of states is realizing that tank water levels usually follow one another fairly closely. Barring a dramatic event, water levels in two tanks in a system are seldom more than a few feet apart. For example if tank A is at 456 ft, tank B will almost always be between 452 and 460 ft even though its possible range may be from 430 to 470 ft. Larger differences usually indicate inadequately sized pipes between the tanks. Therefore, if there are n tanks, the tank levels are divided

5 × (n - 1)

presented in Part IV has [illegible] vicinity water distri- [illegible] To examine the impact [illegible] schedule on the resulting [illegible] applied to both systems for [illegible] savings were [illegible] energy usage cost [illegible] cost savings of [illegible] system.

AD-A189 077 TECHNIQUES FOR IMPROVING ENERGY EFFICIENCY AT WATER SUPPLY PUMPING STATIONS(U) ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MS ENVIR 2/3

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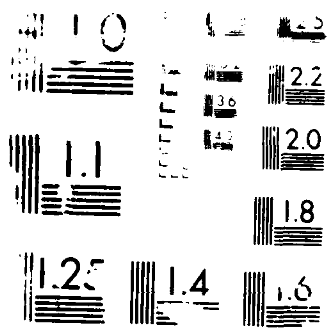
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1.4

PART VI: SUMMARY AND CONCLUSIONS

Summary

97. The purpose of this report was to provide a comprehensive methodology for use in evaluating and improving the overall operating efficiencies of treated water pumping systems. The pump operation methodology has been developed to address three different problems: inefficient pumps, inefficient pump combinations, and inefficient pump scheduling. In order to identify inefficient pumps, pump field test guidelines were developed. An evaluation of the energy consumption and performance of a pump can provide valuable information on its general condition. From this information, a rational decision can be made on the cost-effectiveness of repairing or replacing a low-efficiency pump.

98. To determine optimal pump combinations, a pump combination program was developed. This program uses a set of pump operation curves to determine the best pump combinations possible for a given tank transition (i.e., change in tank water level), system demand, and required pump flow. Methods to develop the pump operation curves are presented in Enclosure 3 of Appendix A.

99. The last problem addressed by the methodology was the problem of inefficient pump scheduling. The current methodology was developed to minimize the electric usage costs for a pumping station. The pump scheduling problem is solved by subdividing the problem into two smaller problems: the optimal pump combination problem and the optimal tank operation problem. The optimal tank operation problem is solved using a tank operation program. This program uses a set of cost operation curves (obtained from successive applications of the PCP) to determine the optimal tank trajectories (i.e., the variation of the tank water levels over time) for a wide range of operating conditions. Once the optimal tank trajectory is obtained, the optimal pump operating policy may be obtained by reapplication of the PCP.

100. As part of this study, the DC and vicinity water system was selected for use as a case study for application of the methodology. In this case the methodology was used to obtain optimal pump operating policies for 4 representative days for two different pressure zones. For the cases examined, average energy usage savings of between 5 and 7.5 percent were obtained. Similar percentage savings can be expected for the low-service and

first high-pressure zones. For the second high-pressure zone, annual energy usage savings of approximately $25,000 were projected. For the third high-pressure zone, annual savings of approximately $65,000 were projected.

101. Before the optimization methodology was applied to the DC system, the pumps in the Dalecarlia and Bryant Street pumping stations were field tested. In general, the majority of the pumps appeared to be operating close to their original design efficiencies. No extremely inefficient pumps or pump combinations were identified.

Conclusions

102. This study has shown that significant energy savings may be achieved through the implementation of policies obtained from the application of optimization procedures. The results of this study can be summarized in terms of three categories of increasing complexity: (a) general rules of thumb, (b) optimal pump combination strategies, and (c) optimal pump operation strategies.

General rules of thumb

103. For the DC and vicinity distribution system, the following general rules of thumb are suggested:

a. During the critical demand month (usually August or September), the pumps should be operated at a fairly constant rate to minimize the electrical distribution charge.

b. In general, the demand load for the system should be equally shared between both pumping stations (especially during the critical demand month). Such a policy should lead to a further minimization of the electrical distribution charge.

c. During the weekends, when the energy usage rate is constant, the pumps should be operated so that the tanks fill gradually.

d. During the weekdays, when a variable energy usage rate is in effect, the pumps should be operated such that the tanks are filled during the off-peak (8 pm to 8 am) and intermediate (8 am to noon) periods. During the on-peak periods (12 noon to 8 pm), as few pumps as possible should be operated.

Optimal pump combination strategies

104. To obtain optimal pump combination strategies for each pressure zone, an optimal pump combination program was developed. For a specified system demand and a desired change in tank level, the PCP can be used to rank

the feasible pump combinations on the basis of economic efficiency. From this ranking, the operator can select the combination that is both economical and practical. The PCP is very practical and can easily be incorporated into the daily operation of each pumping station. In applying the program to the DC system, the program was able to simulate the hydraulics and economics of the system very accurately.

Optimal pump operation policies

105. The development of optimal pump operation policies requires the application of a sophisticated optimization program along with the PCP. Application of these programs to the second and third high-pressure zones resulted in significant energy savings. Additional data and software will be required to develop and implement such policies for the entire system on a daily basis. However, given the results of the present study, such an effort would appear to be economically promising.

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APPENDIX A: ENERGY EFFICIENCY AT WATER SUPPLY PUMPING STATIONS

1. As part of the work unit under which this report was prepared, the authors developed an engineer technical letter (ETL 1110-1-134) for use by Corps of Engineer personnel. This ETL, reproduced on the following pages, provides additional guidance on studies conducted to improve water distribution pumping efficiency.

DEPARTMENT OF ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

ETL 1110-1-

DAEN-ECE-B

Engineer Technical
Letter No. 1110-1-

Engineering and Design
ENERGY EFFICIENCY
AT WATER SUPPLY PUMPING STATIONS

1. Purpose. This letter describes techniques for evaluating the efficiency of existing pumping stations, selecting efficient pump combinations, and optimally scheduling pumping with the goal of reducing energy costs. This should assist in identifying inefficient pumps, combinations of pumps, and operating policies.

2. Applicability. This letter is applicable to Corps-operated pumping facilities such as the Washington Aqueduct Division and water supplies at recreation areas, FOA's conducting specially authorized water supply studies or Section 22 (PL 93-251) studies, and FOA's conducting design work or performing other services at military installations. Energy conservation is an important consideration at both civil (ref 3b) and military (ref 3c) projects.

3. References.

a. Section 22 of PL 93-251, Water Resources Development Act of 1974.

b. ETL 1110-2-216, Energy Conservation for Civil Works.

c. ETL 1110-3-282, Energy Conservation.

d. Ormsbee, L. E., and Walski, T. M. "Techniques for Improving Energy Efficiency at Water Supply Pumping Stations," Technical Report EL-87-X, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

4. Background. Water supply pumps can account for a great deal of the energy consumed by a municipality or military installation. One of the largest energy use in the Corps of Engineers is the pumps at the Corps' Washington Aqueduct Division. Neither the civil (ref 3b) nor the military (ref 3c) guidance on energy consumption discusses water pumping or potential energy savings in water pumping. Pumps can wear out, carrying capacity can be lost in suction and discharge piping, pumps may have been improperly sized originally, pumps can be operated inefficiently, or equalizing storage capacity may be inadequate. Any of the above can result in wasted energy. Pumps should be evaluated periodically to ensure they are operating efficiently. This letter describes ways to evaluate pump operation and identify inefficiencies.

5. Overview. Several types of analysis can be performed to evaluate pump energy uses, ranging from determination of the efficiency of an individual pump to examination of operating policies with the option of adding distribution storage. These techniques are described in four enclosures to this letter as described below:

a. Pump System Fundamentals. Enclosure 1 is a review of fundamental concepts of pumping system design and operation and is intended as a review for practicing engineers.

b. Field Testing Individual Pumps. The most fundamental type of test that can be conducted is that of measuring the efficiency of an individual pump. The overall (wire-to-water) efficiency is the ratio of water horsepower produced by the pump to the input horsepower, usually electrical. The efficiency should be measured at several flow rates. These tests will identify pumps that fail to meet performance specifications, as well as the most efficient operating flow rate for each pump. Procedures for conducting these tests are described in Enclosure 2. Measurement of pump flow rate can be very difficult or impractical, depending on the size of the pump and the configuration of the intake and discharge channels. Therefore a determination should be made at an early stage to determine if it is practical to make efficiency tests.

c. Multiple Pump Operation. The fact that a pump meets its original performance specifications does not guarantee that it will operate efficiently. A pump can operate over a wide range of flow rates. The actual flow that it produces depends on the head difference between the suction and discharge sides of the pump. The relationship between these heads and the flow rate is referred to as the system head curve and is a function of tank water levels on each side of the pump, pipe carrying capacity near the pump, location of water use with respect to the pumps, and which other pumps are operating. Depending on the system head encountered by a pump, the pump may perform over a wide range of efficiencies. Enclosure 3 describes how to identify which pump or combination of pumps is most efficient for a given system head.

d. Pump Operating Policies. Given that a pumping station must produce a specific volume of water on a certain day, the operating policy to produce that volume can vary from operating the pumps at a constant flow rate, to producing a flow rate that follows demand, to pumping at a higher rate when off-peak energy prices are in effect. The best operating policy depends on time of day pricing schedule, the water demands as a function of time, the amount of storage available, the efficiencies of the individual pumps, and the carrying capacity of the distribution system. Selecting the optimal operating policy requires using a computerized procedure because of the complexity of the problem. A dynamic programming model to solve this type of problem was developed by Ormsbee and Walski of the US Army Engineer Waterways Experiment Station (ref 3d) and is described in Enclosure 4.

6. Action. This techniques described in this letter should be used by FOA's in studies involving water supply pumping design and operation.

FOR THE COMMANDER:

4 Encl
as

PUMP SYSTEM FUNDAMENTALS

1. Introduction. Energy costs comprise the major component of the operating costs of most utilities. The largest quantity of energy is usually consumed by the treated water pumping units. The operation cost associated with a particular pump station will be dependent upon four different factors: the pumps, the distribution system, the pump drivers (motors), and the governing energy rate schedule. This enclosure examines the characteristics of each of these components and their influence on the operational efficiency of a water supply system.

Section I: Pump Characteristics

2. Centrifugal Pumps. By far, the most commonly used pump in water supply is the centrifugal pump. Centrifugal pumps add energy to the flow through the use of an impeller. Centrifugal pumps can be classified into three general categories according to the way the impeller impacts energy to the fluid. Radial flow impellers impart energy primarily by centrifugal force. Liquid enters the impeller at the center and flows radially to the outside of the pump casing. Mixed flow impellers impart energy partially by centrifugal force and partially by axial force, since the vanes of the impeller are acting partly as an axial compressor. Axial flow impellers impart energy to the fluid by acting as axial flow compressors. In axial flow pumps, fluid enters and exits along the axis of rotation. All three types of centrifugal pumps are used in water supply applications. In general, radial flow impellers are used in high head pumps while axial flow impellers are used in lower head pumps. In this ETL, only constant-speed centrifugal pumps will be considered.

a. Specific Speed. The particular type of pump required for a given application can usually be found by determining the specific speed of the pump. The specific speed may be defined as follows:

$$N_s = \frac{N\sqrt{Q}}{H^{0.75}} \qquad (1\text{-}1)$$

where N_s = specific speed

N = pump speed, rpm

Q = discharge, gpm

H = pump head, ft

In determining the specific speed of a pump, H and Q are measured at the point of maximum efficiency (Medcaff and Eddy 1972). In general, the computed value of the specific speed has no usable physical significance, but it is useful because it can be used as a guide in selecting the most efficient pump type. Generally, pumps with low specific speeds (500 to 2,000 rpm) are made

ENCLOSURE 1

to deliver small discharges at high pressures. Pumps characterized by high specific speeds (5,000 to 15,000 rpm) deliver large discharges at low pressures (Simon 1976). For centrifugal pumps, the value of specific speed can be used to select pumps.

Table 1-1. Specific Speed Versus Impeller Type

Range of Specific Speed, rpm	Impeller Type
500 - 3,000	Radial Vane
4,000 - 7,000	Mixed Flow
9,000 and above	Axial Flow

b. Net Positive Suction Head. Water is not sucked into a pump. Instead, a positive head must push the liquid into the pump. The Net Positive Suction Head (NPSH) is the total head (in feet of liquid) on the suction side of a pump less the absolute vapor pressure (in feet) of the liquid being pumped. In order for cavitation not to occur, the Net Positive Suction Head Available (NPSHA) must be greater that the Net Positive Suction Head Required (NPSHR) (Lindeburg 1981).

(1) NPSHR. NPSHR is determined by the pump manufacturer and will depend on many factors, including the type of impeller inlet, impeller design, pump flow, pump speed, the nature of the fluid, etc. The NPSHR for a particular pump is usually plotted on the pump performance curve as a function of discharge.

(2) NPSHA. NPSHA is the net positive suction head that is available in the field for a set of particular operating conditions. NPSHA can be calculated or can be obtained by measuring pressure (or vacuum) at the suction side of the pump. For negative suction lift conditions, NPSHA may be determined using Equation 1-2. For positive suction conditions, NPSHA may be determine using Equation 1-3.

Negative Suction Conditions: $NPSHA = h_a - h_{vpa} - h_{st} - h_{fs} - h_{vh}$ (1-2)

Positive Suction Conditions: $NPSHA = h_a - h_{vpa} + h_{st} - h_{fs} - h_{vh}$ (1-3)

where h_a = absolute pressure (in feet of liquid) on the surface of the liquid supply level (this will be barometric pressure if suction is from an open tank or sump; or the absolute pressure existing in a closed tank), ft

h_{vpa} = the head corresponding to the vapor pressure of the liquid at the temperature being pumped (at 20° C the vapor pressure of water at sea level ft), ft

h_{st} = static height that the liquid supply level is above or below the pump center line, ft

h_{fs} = all suction line losses, including entrance losses and friction losses through pipe, valves, and fittings, etc.

h_{vh} = velocity head, ft

Usually velocity head and suction line losses are negligible.

c. Pump Performance Curves. For any centrifugal pump, several curves can be developed to show the relationships between flow rate, head, NPSHR, horsepower, and efficiency. Performance curves are usually plotted with flow (pump discharge) on the horizontal axis and the other characteristics plotted on the vertical axis as shown in Figure 1-1. The characteristic curves for a constant-speed centrifugal pump is based on a certain speed, impeller diameter, and fluid viscosity.

(1) Head-discharge curves. The head-discharge curve indicates the relationship between the head or pressure developed by the pump and the corresponding flow rate through the pump. In most cases the flow rate decreases continuously with increasing head as shown in Figure 1-1. As the head increases, the flow rate decreases to zero. The head at which zero flow occurs is known as the cutoff head. As the head-discharge curve approaches maximum flow, the velocity in the impeller eye may become so high that the head-discharge curve will drop abruptly in a vertical direction. The point where this drop occurs is known as the cutoff discharge. The point corresponding to the pump's maximum efficiency (Best Efficiency Point, BEP) is usually indicated at shown in Figure 1-1.

(2) NPSHR-discharge curves. The NPSHR-discharge curve indicates the required net positive suction head required in order to prevent cavitation for a given pump discharge. The NPSHR-discharge curve is a characteristic of the pump and must be obtained from the manufacturer.

(3) Water horsepower-discharge curves. Water horsepower is defined as the power that is delivered by the pump to the fluid it is pumping. The water horsepower-discharge curve will generally slope upward from left to right until a maximum is achieved and then slope downward. The water horsepower-discharge curve may be constructed using the following relationship:

$$WHP = \frac{Qh\gamma}{550} \qquad (1\text{-}4)$$

where WHP = water horsepower

Q = discharge, cfs

h = pump head, ft

γ = specific weight of fluid (62.4 lb/ft^3 for water)

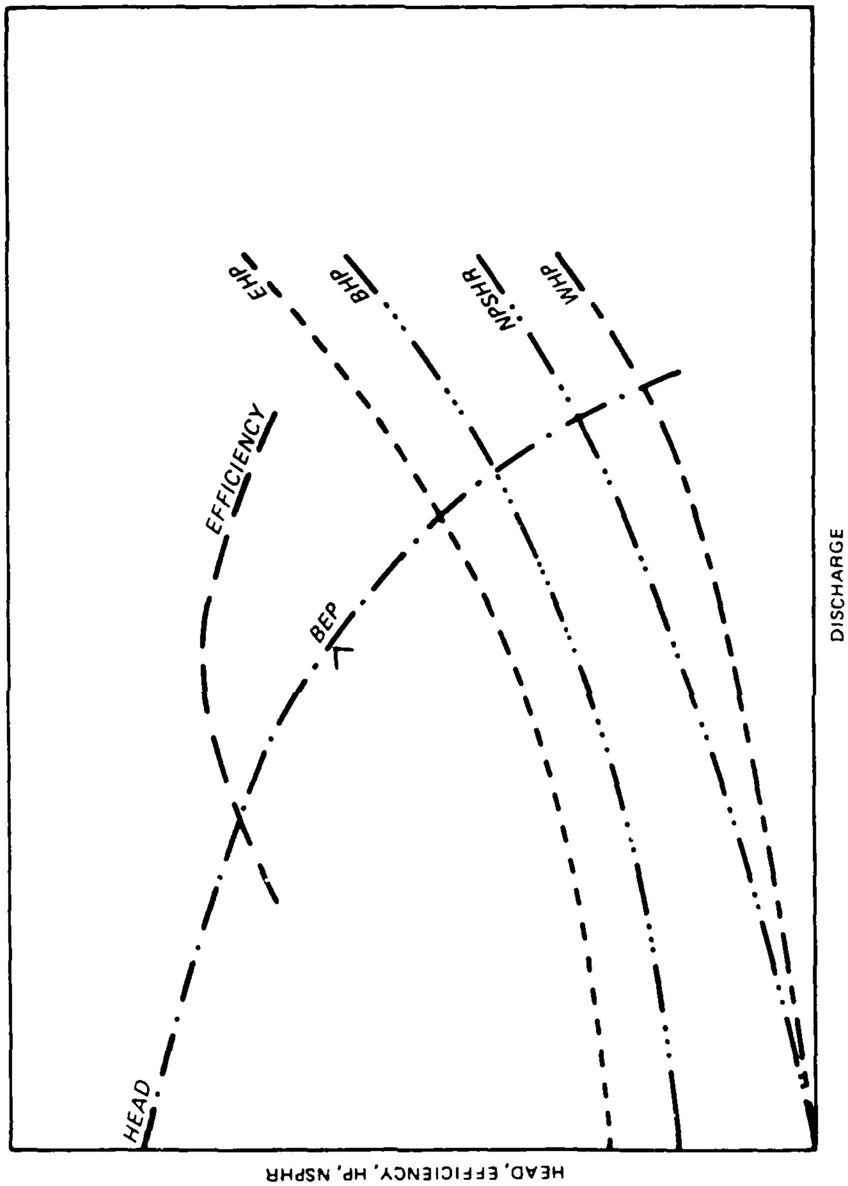

Figure 1-1. Pump Characteristic Curves

(4) Brake horsepower-discharge curves. Brake horsepower is defined as the power required to drive the pump. Brake horsepower can only be measured using special instrumentation, and it is usually not measured in the field. If the efficiency of the driver (e_d) is known, the brake horsepower may be determined from the following equation:

$$BHP = EHP * e_d \qquad (1\text{-}5)$$

where BHP = brake horsepower

EHP = electrical horsepower

e_d = driver efficiency

The brake horsepower-discharge curve for a particular pump generally slopes upward from left to right in the opposite direction from the head-capacity curve (see Figure 1-1). At very high flow, the brake horsepower-discharge curve may reach a maximum and then slope slightly downward.

(5) Electric horsepower-discharge curves. Electric horsepower (also called wire horsepower) is defined as the power required to drive the motor (driver) that is turning the impeller in the pump. Electrical horsepower may be determined by measuring the electrical power used by the motor:

$$EHP = \frac{KW}{0.746} \qquad (1\text{-}6)$$

where EHP = electrical demand, horsepower

KW = electrical demand, kilowatts

The head-electric horsepower curve will also slope upward from left to right as shown in Figure 1-1. The electric horsepower-discharge curve for a particular pump-driver combination will be higher than the brake horsepower-discharge curve because of inefficiency of the driver.

(6) Pump efficiency curves. The pump efficiency curve will normally rise gradually from left to right to a maximum at its best efficiency point and then drop off as the head begins to decrease much more rapidly than the discharge increases (see Figure 1-1). The pump efficiency (e_p) is defined as the ratio of the water horsepower (WHP) to the brake horsepower (BHP):

$$e_p = \frac{WHP}{BHP} \qquad (1\text{-}7)$$

(7) Driver efficiency curves. The driver efficiency curve is usually fairly constant for most motors. The driver efficiency (e_d) is defined as the ratio of the brake horsepower (BHP) to the electrical horsepower (EHP):

$$e_d = \frac{BHP}{EHP} \qquad (1\text{-}8)$$

(8) Wire-to-water efficiency curves. When evaluating the overall efficiency of a pump-driver installation, the most useful efficiency is the overall (or wire-to-water) efficiency (Walski 1984). The wire-to-water or overall efficiency (e_w) is defined as the ratio of the water horsepower (WHP) to the electric horsepower (EHP):

$$e_w = \frac{WHP}{EHP} \tag{1-9}$$

Alternatively, the overall efficiency may be expressed as the product of the pump efficiency and the driver efficiency:

$$e_w = e_p * e_d \tag{1-10}$$

The wire-to-water efficiency curve will normally rise gradually from left to right to a maximum at its best efficiency point and then drop off as the head begins to decrease much more rapidly than the capacity increases (see Figure 1-1).

Section II: System Characteristics

3. <u>Pump Operating Point</u>. Although the pump characteristic curves show the relationship between head, flow rate, and efficiency over a wide range of possible operating conditions, they do not indicate the point on the curve at which the pump will be operating for a particular piping system. The operating point is found by determining the intersection of the head versus discharge curve with what is called the system head curve of the distribution network.

4. <u>System-Head Curve</u>. The system head curve for a particular distribution network represents the total head against which a particular pump or group of pumps will have to operate as a function of flow rate. In order to develop the system head curve for a particular system, the distribution network is usually idealized as shown in Figure 1-2. The idealized system consists of two tanks (the clearwell and the controlling elevated storage tank), two pipes (the suction pipe and the pipe to the storage tank), and the pump. The system curve may be developed by writing an energy equation between the two tanks and plotting the total energy difference as function of discharge. The system head curve consists of two components: the static head and the friction head (see Figure 1-3). The relationship between the system head and discharge may be expressed as:

$$h_{sys}(Q) = h_s + h_f \tag{1-11}$$

where $h_{sys}(Q)$ = system head as a function of Q

h_s = static head

h_f = friction head

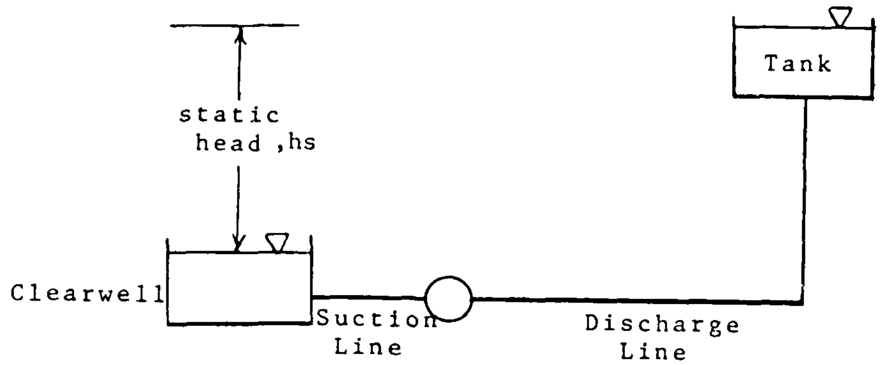

Figure 1-2. Simplified Network

a. Static Head. The static head (h_s) is the difference in the water surface elevations of the two tanks. Since the water level in the wet well and the water level at the discharge point may vary, the static head will also vary, resulting in a family of parallel curves corresponding to different values of h_s .

b. Friction Head. The friction head is equal to the head loss between the two tanks. For the idealized system shown in Figure 1-2, the friction head may be expressed as:

$$h_f(Q) = K_s Q_s^n + K_d Q_d^n \tag{1-12}$$

where $h_f(Q)$ = friction head, ft

K_s = suction line head loss coefficient

Q_s = flow rate in suction line, gpm

K_d = discharge line head loss coefficient

Q_d = flow rate in discharge line, gpm

For the idealized case, Q_s and Q_d are equal to the pump discharge. The exact values for K and n will depend on the type of head loss equation employed. For the Hazen-Williams head loss equation, $n = 1.873$ and K may be expressed as follows:

$$K = \frac{4.73L}{C^{1.852} \quad D^{4.87}} \tag{1-13}$$

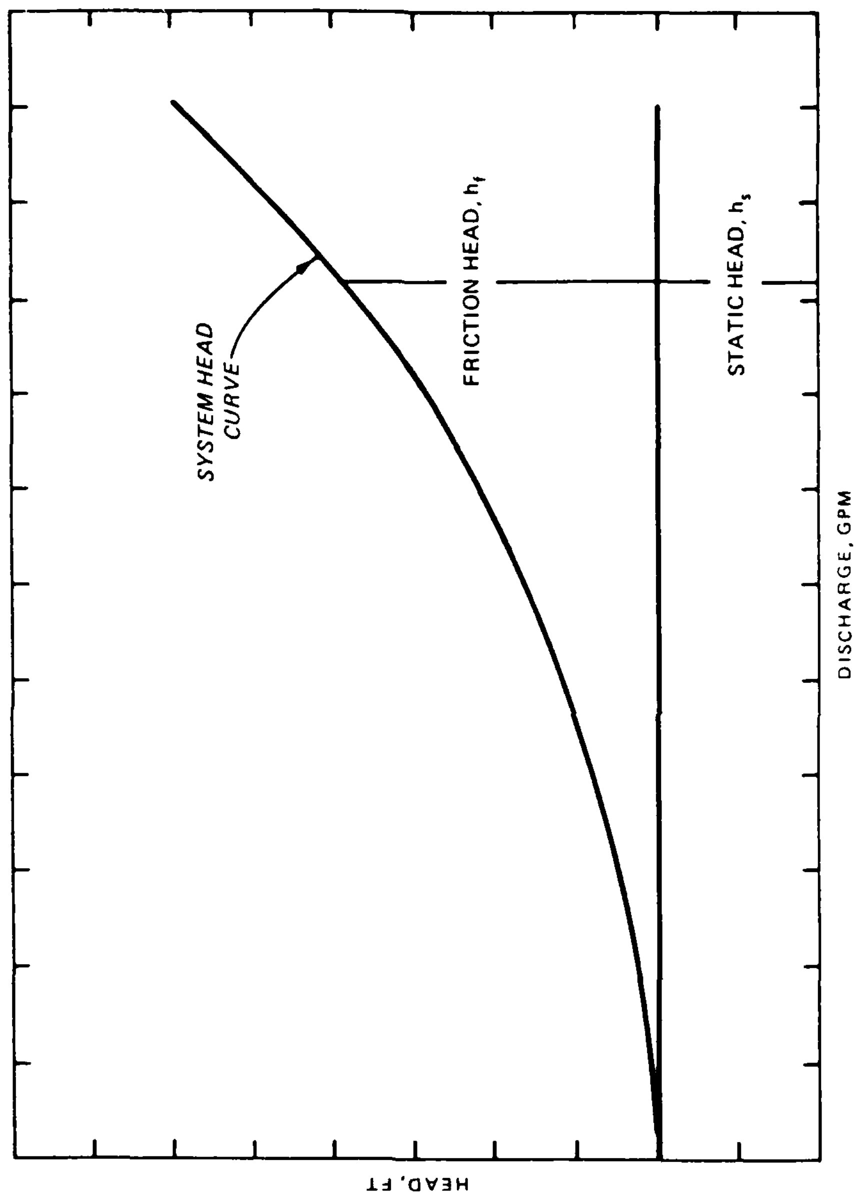

Figure 1-3. System Head Curve

where L = pipe length, ft

C = Hazen-Williams coefficient

D = pipe diameter, ft

For systems with very short lift lines, minor losses can be important. For these types of systems, all minor losses may be converted into equivalent lengths of pipe through the use of standard equivalent length tables. Head loss through the equivalent length of pipe can then be determined using the Hazen-Williams equation.

5. Example. Consider the idealized network shown in Figure 1-2. The characteristic curve for the pump is shown in Figure 1-4. Develop a system head curve for this system and determine the operating point for the pump for the following conditions using the following data:

Clearwell Elevation = 100 ft	Tank Elevation = 300 ft
L_{suc} = 100 ft	L_{dis} = 10,000 ft
D_{suc} = 12 in.	D_{dis} = 12 in.
C_{suc} = 120	C_{dis} = 120
$K_{suc} = 0.067 = \dfrac{4.73(100)}{120^{1.852}\ (12/12)^{4.87}}$	K_{dis} = 6.671
$n = 1.852$	$n = 1.852$

a. Static Head. $h_s = 300 - 100 = 200$ ft

b. Friction Head. $h_f = K_s Q_s^n + K_d Q_d^n = 6.74\ Q^n$

c. Total Head. $h = 200 + 6.74\ Q^n$

d. System Head Curve. The system head curve may be obtained by substituting values for flow rate (cfs) into the total head equation above and solving for head as shown in the following table.

Q (gpm)	Q (cfs)	H (feet)
0	0	200
500	1.116	208
1,000	2.232	230
1,500	3.348	264

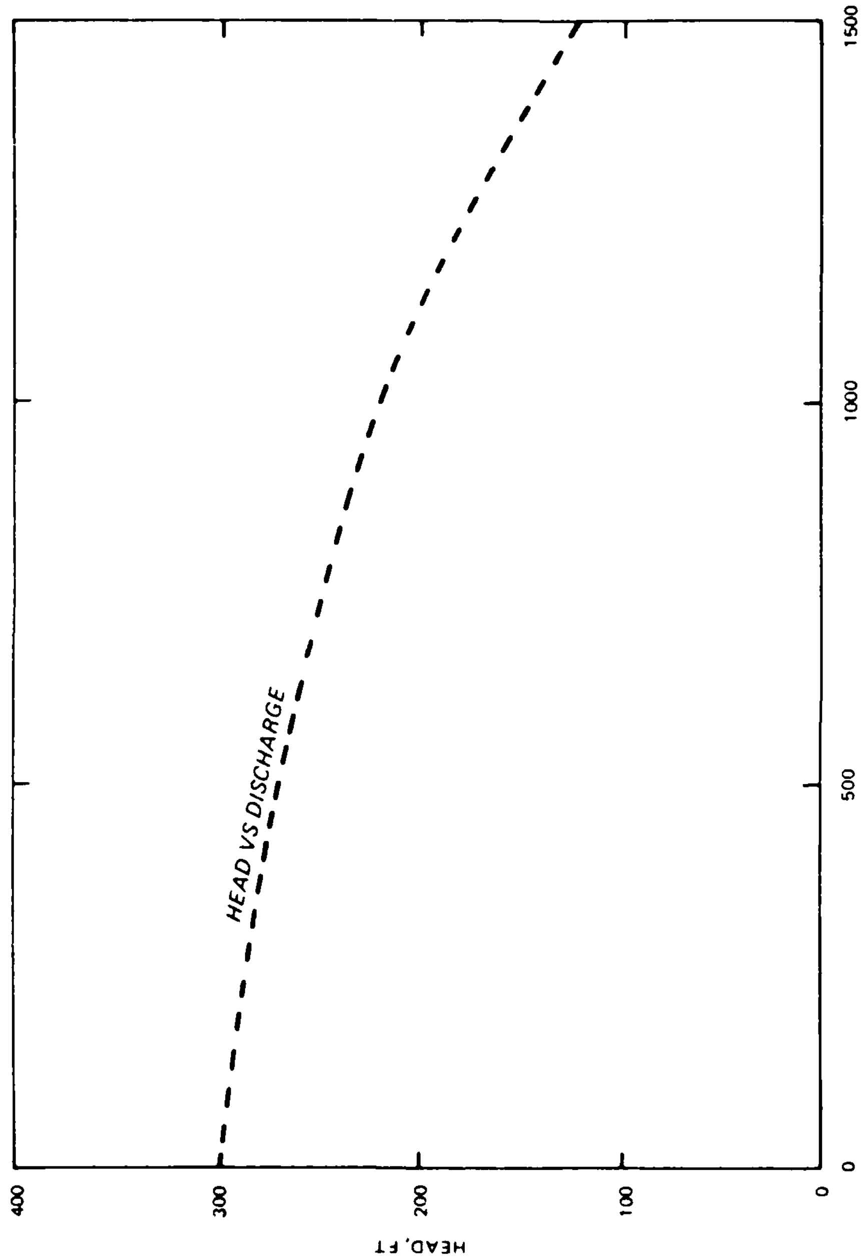

Figure 1-4. Example Head Versus Discharge Curve

The pump operating point for a given static head may be obtained by superimposing the system head curve (developed above) on the head versus discharge curve (Figure 1-4) as shown in Figure 1-5. In this problem, the pump will produce 1,000 gpm at 230 ft of head. Ideally, the operating point should be near the point of maximum pump efficiency.

6. Network Idealization. For single pump stations with single controlling discharge elevations (e.g., a single elevated storage tank), the energy equation may be written from the clearwell of the pump to the elevation of the controlling discharge point (normally a water level in an elevated tank). For a system with more than one tank, the controlling tank can usually be determined by writing equations between both tanks and then selecting the more critical curve. These system curves are based on the assumption that there is virtually no water used or lost between the two tanks. This is a reasonable assumption for many water supply pumps. However, in some water distribution systems, very large quantities of water can be consumed between the pump and the nearest tank. In such cases, the system head curves will be affected. In order to account for these effects it may be necessary to perform a simplified network analysis of the system to generate the system head curve (Ormsbee and Walski 1986).

Section III: Driver Characteristics

The majority of centrifugal pumps are driven by squirrel cage induction motors. For large, low-speed drives, synchonous motors may be used. Both types of motors are discussed briefly in the following sections (Andreas 1982).

7. Induction Motors. The line current (I_λ) drawn by induction motor consists of two components: the reactive current (I_μ) and the power-producing current (I_ρ).

a. Reactive Current. The reactive current is that current required to produce the magnetic flux in the motor. This component of current creates a reactive power requirement that is measured in kilovolt-amperes reactive (kilovars, kvar).

b. Power-Producing Current. The power-producing current is that current which reacts with the magnetic flux to produce the output torque of the machine. This component of current creates the load power requirements measured in kilowatts (kW).

c. Total Line Current. The total line current drawn by an induction motor is the vector sum of the reactive current and the power-producing current. The vector relationship between the line current (I_λ) and the reactive component (I_μ) and load component (I_ρ) currents may be expressed by a vector diagram as shown in Figure 1-6, where the line current I_λ is the vector sum of the two components.

d. Total Apparent Power. In the same way that the line current was related to the magnetizing current and the power-producing current, the total apparent power (kVA) may be related to the kilovar power (kvar) and the

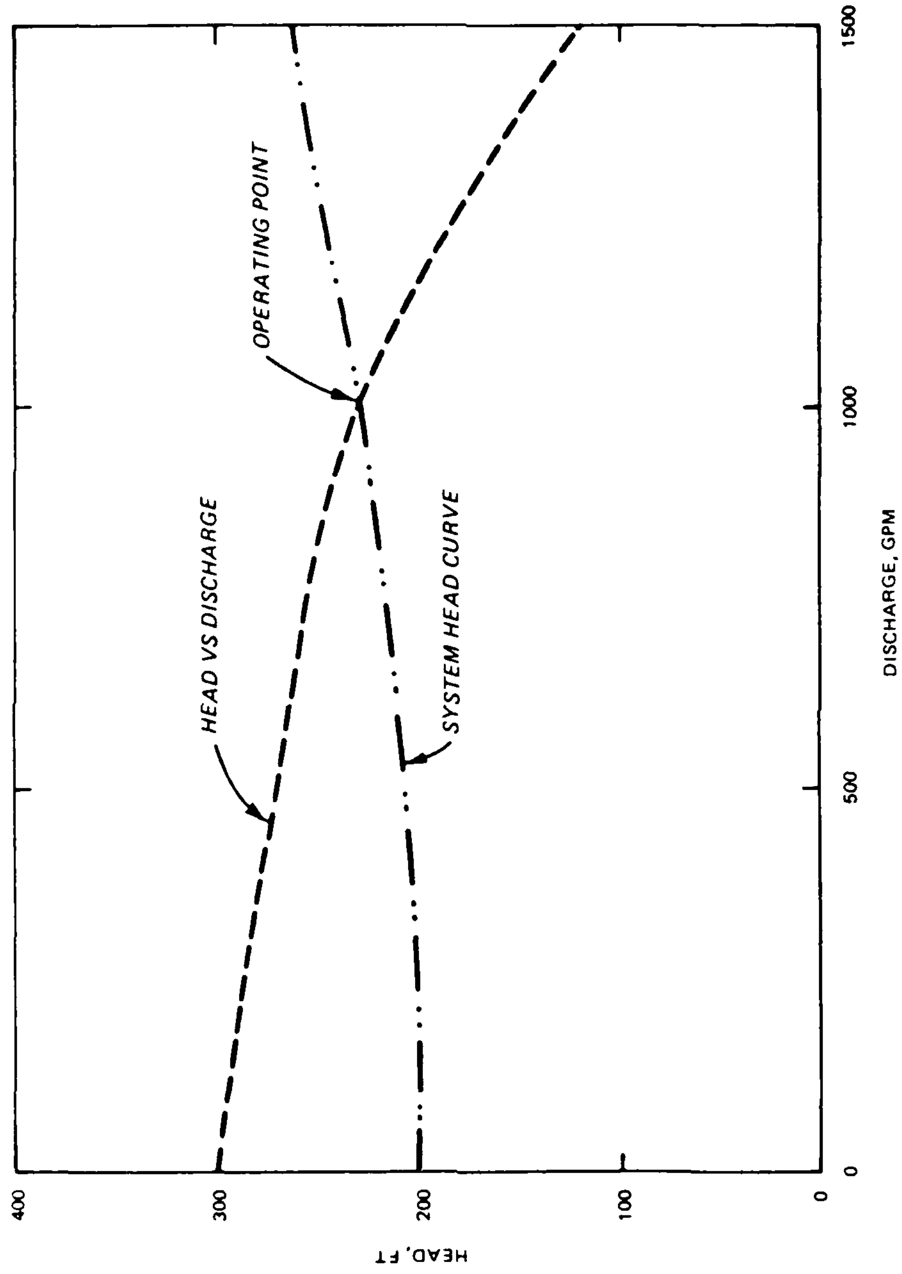

Figure 1-5. Pump Operating Point

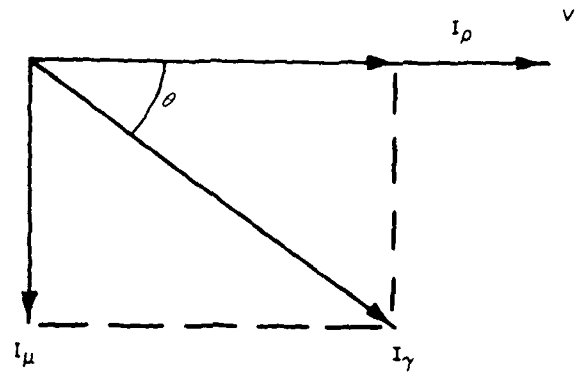

Figure 1-6. Current Vector Diagram

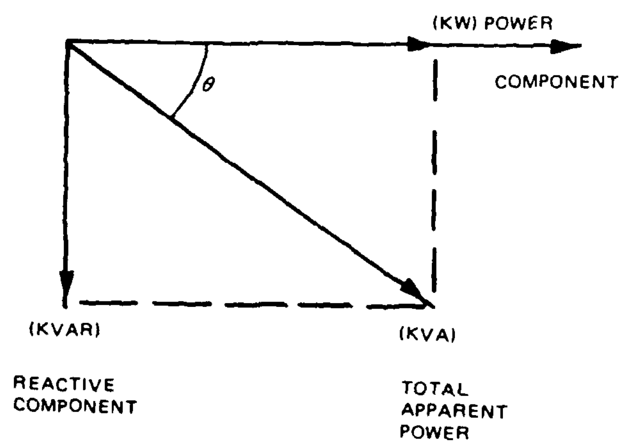

Figure 1-7. Power Vector Diagram

kilowatt power (kW) as shown in Figure 1-7. For three phase motors, the apparent power or kVA input to a motor may be expressed as:

$$kVA = I_\lambda V_\lambda \sqrt{3} \div 1{,}000 \qquad (1\text{-}14)$$

where I_λ = total line current, amps

V_λ = voltage, volts

kVA = total apparent power, watts.

e. Reactive Power. The reactive power is the power that must be supplied by the electric utility company to produce the reactive component of the total line current. Since the reactive current is 90 electrical degrees out of phase with the applied voltage, it does no work and thus cannot be measured with a standard kW meter.

f. Load Power. Most electric utility rates are based on the load power (kW) instead of the total apparent power (kVA). If the reactive power (kvar) and the apparent power (kVA) are available, the corresponding load power may be obtained as follows:

$$kW = \sqrt{(kVA)^2 - (kvar)^2} \qquad (1\text{-}15)$$

where kW = load power, watts

kvar = reactive power, watts

8. Power Factor. The power factor of a motor is defined as the cosine of the angle θ between the line current and the voltage. This vector relationship can also be expressed in terms of the components of the total kVA input, and is shown in Figure 1-7. In this case, the power factor is the cosine of the angle θ between the total kVA and kW inputs to the motor. The system power factor can be determined by a power factor meter reading or by the ratio of the load power (kW) to the total apparent power (kVA). Thus,

$$\text{Power Factor} = \frac{kW}{kVA} \qquad (1\text{-}16)$$

Theoretically, the power factor may vary from 0 to 100 percent. A low power factor causes poor electrical system efficiency. The total apparent power must be supplied by the electric utility. With a low power factor, or a high kvar component, additional generating losses occur throughout the system. To discourage low power factor loads, most utilities impose some form of penalty or charge in their electric power rate structure for a low power factor. The power factor for most induction motors ranges from 50 to 90 percent. (Andreas 1982).

9. Example. The total line current to a pump driven by a squirrel cage induction motor is measured to be 200 amps. If the line-to-line voltage is measured to be 2.4 kV and the reactive component of the total apparent power is measured to be 400 kvars, determine the kW power component and the power factor of the motor as follows:

$$kVA = (200 \text{ amps}) * (2{,}400 \text{ V}) * \sqrt{3} / 1{,}000 = 831$$

$$kvar = 400 \text{ (given)}$$

$$kW = \sqrt{(831)^2 - (400)^2} = 728$$

$$\text{[illegible]} = kW/kVA = 728/831 = 0.88$$

10. Synchronous Motors. To improve the power factor for a given load, the reactive load component (kvar) must be reduced. This component of reactive power lags the power component (kW input) by 90 electrical degrees. One way to reduce the effect of this component is to introduce a reactive power component that leads the power component by 90 degrees. This can be accomplished by using synchronous motors. Synchronous motors can be adjusted to operate at a leading power factor, thus providing leading kvars to offset the lagging kvar of inductive-type loads such as induction motors. As a result, synchronous motors can be used to decrease or eliminate the power factor cost associated with the operation of a particular pump. Synchronous motors can be used to improve the overall cost efficiency of a pump, but such motors are generally more expensive than standard induction motors. As a result, synchronous motors are normally used only for applications where larger horsepowers are needed or where power factor correction is important (Andreas 1982).

Section IV: Electric Rate Schedules

Electrical utility company rate schedules applicable to pumping units are usually divided into three different components: (a) energy consumption charge, (b) demand charge, and (c) power factor charge (Reid 1980). Each of these charges is discussed below:

11. Energy Consumption Charge. The energy consumption charge is that portion of the electric utility bill based on the kilowatt-hours of electric energy consumed during the billing period. In many instances, the energy consumption charge is based on a declining block rate (Andreas 1982). As the total consumption increases, the rate decreases. For example:

First 50 kWh: $3.60 flat charge

Next 450 kWh: $0.0621/kWh

Next 14,500 kWh: $0.0521/kWh

Over 15,000 kWh: $0.0231/kWh

Alternatively, the rates may be different depending upon the time of day or the season of the year. For example:

Time Period	Summer	Winter
12 midnight to 8 a.m.	$0.0295/kWhr	$0.0205/kWhr
8 a.m. to 12 noon	0.0462	0.0395
Noon to 8 p.m.	0.0624	0.0523
8 p.m. to midnight	0.0462	0.0395

In some cases the energy charge may be a flat rate per kilowatt-hour of consumption (Patton and Horsely 1980).

12. Demand Charge. The demand charge is usually based on the maximum power demand in kilowatts during a specified time period, frequently 1 year. The time interval for determining the kilowatt demand is usually 15 to 30 minutes. As with the energy consumption charge, the demand charge may vary depending upon the time of day or the season of the year. The demand charge is generally a function of an electric utility's fixed costs and the expenses for generating plants, transmission lines, substations, and other items required to satisfy peak loads on the system (Andreas 1982). It is usually given in dollars per kilowatt.

13. Power Factor Charge. The power factor charge is used by some utilities to compensate for the increased cost of supplying energy to consumers with certain electrical load characteristics. Induction motors commonly used to drive water pumps can exhibit low power factors which may result in power factor charges. Although the power factor charge may be treated as a separate charge, it is more commonly treated as an adjustment to the demand charge (Andreas 1982).

Section V: Pump Efficiency

The overall energy cost associated with a pump station may be reduced by decreasing any one of the three standard energy rate components discussed in the previous sections.

14. Power Factor Charge Reduction. The power factor charge can be decreased by improving the power factor associated with a particular pump. The power factor can be improved by changing the motor load or the motor type, or through the use of capacitors (Patton and Horsley 1980).

15. Energy Consumption Charge Reduction. The total energy consumption charge associated with a particular pump station can be decreased by improving the operational efficiencies of each pump. In order to evaluate the existing efficiency of a pump, it should first be field tested. Guidelines for field testing pumps and evaluating the resulting efficiencies are provided in Enclosure 2. Pumps with low efficiencies can be reconditioned or replaced. For pump stations with multiple pumps, the energy consumption charge may also be reduced by operating the multiple pumps at efficient combined operating points. Guidelines for determining the optimal operating points for multiple pumps are given in Enclosure 3. When time of day energy pricing is used, pumping cost

can be reduced by pumping more during off-peak hours. This type of evaluation is described in Enclosure 4.

16. Demand Charge Reduction. Although the total energy consumption charges associated with a pump operation can be decreased by evaluating the operational efficiencies of a single pump or multiple pump combinations, such a program may have little impact on electrical demand charges. The reduction of electrical demand charges usually requires modifications of the pump station operating procedures which would include operating pumps at a fairly constant discharge and installing additional elevated storage to meet peak demands (Lackowitz and Petretti 1983). Guidelines for evaluating and improving the operational procedures of a particular pump station are given in Enclosure 4.

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FIELD TESTING INDIVIDUAL PUMPS

1. Introduction. Although pump characteristic curves for a particular pump can usually be obtained directly from the manufacturer, the actual field performance may not correspond to the design engineer's expectations. Because of this, all pumps should be field tested as a first step in any comprehensive pump efficiency study. At a minimum, this will involve measuring discharge, head, and energy consumption, in order to calculate efficiency when the pump is operating. More thorough testing would involve generating system head curves. Pump tests or pump model tests conducted by the pump manufacturer, as part of the pump supply contract and witnessed by the Government, in general, are more accurate and less expensive than field tests.

2. Test Standards. Various standards have been developed by the Hydraulic Institute (1983) and AWWA (1983) for use in testing pumps. General guidelines for constructing the pump characteristic curves for constant-speed centrifugal water supply pumps are provided in the following sections.

Section I: General Guidelines

3. Pump Characteristic Curves. The basic objective of most pump field tests is the reconstruction of the pump characteristic curves from field data. In general, two different pump characteristic curves are necessary to evaluate the performance of a pump: the head versus discharge curve, and the wire-to-water efficiency versus discharge curve. The head-discharge curve can be constructed directly from field measurements of pressure and flow. The wire-to-water efficiency (e_w) versus discharge curve can be constructed form the water horsepower (WHP) versus discharge curve and the electrical horsepower (EHP) versus discharge curve using the following equation:

$$e_w = \frac{WHP}{EHP} \qquad (2\text{-}1)$$

where e_w = wire-to-water efficiency

WHP = water horsepower, h_p

EHP = electrical horsepower, h_p

The water horsepower versus discharge curve can be constructed directly using the data from the constructed head-discharge curve and the following equation:

$$WHP = \frac{Qh_p\gamma}{550} \qquad (2\text{-}2)$$

where WHP = water horsepower

Q = discharge, cfs

ENCLOSURE 2

h_p = head, ft

γ = specific weight of fluid

= 62.4 lb/ft^3 for water

The electrical horsepower versus discharge curve may be constructed directly from the test data using the following equation:

$$EHP = \frac{kW}{0.746} \tag{2-3}$$

where EHP = electric power, hp

kW = electric power, kilowatts

In summary, both the head versus discharge and the efficiency versus discharge curves can be constructed from three different types of data: pressure (head) readings, flow readings, and power readings. All three types of data can be obtain during a standard pump field test.

4. Test Procedure. The general procedure for field testing a pump involves measuring the head on the pump and the corresponding flow rate and electrical horsepower. The discharge head on the pump may be changed by throttling valves downstream of the pump. In order to collect the necessary data, the pump is usually started with the discharge valve closed. This condition allows a head reading for the cutoff head of the pump. After this head has been determined, the discharge valve is then slowly opened in several discrete steps. After each step, a few minutes should be provided for transient effects to dampen. Once steady conditions have been obtained, the discharge and electrical horsepower readings corresponding to the measured head can be obtained. After the valve has been completely opened, the valve can be slowly closed using the same discrete steps as before. This procedure will thus provide two sets of readings for each test.

5. Multiple Pumps. When multiple pump stations are being tested, one pump may be fully opened and others may be added successively to determine the total flow from the pump stations as each unit is brought online. In this manner, the incremental flows that the units contribute to the distribution system can be established and the inefficiencies involved under specific operating conditions can be identified.

6. Test Preparation. Before conducting any field test, it is best to visit the pump station beforehand to determine what type of gages are available and what type of testing equipment may be needed. In addition, it is usually helpful to prepare a data sheet for use in recording and reducing the test data for each pump. A typical data sheet is shown as Figure 2-1, along with the equations needed to reduce the data.

PUMP CALIBRATION TEST		PUMP STATION	
DATE	______	TIME	______
PUMP NUMBER	______	CLEARWELL ELEV	______
MODEL NUMBER	______	PUMP ELEV	______
SERIAL NUMBER	______	TANK ELEV	______
RATED DISCHARGE	______	RATED HEAD	______
RATED SPEED	______	RATED POWER	______

MEASURED QUANTITIES				COMPUTED QUANTITIES			
	UPSTREAM	DOWNSTREAM	ELECTRICAL	(1)	(2)	(3)	(4)
FLOW	PRESSURE	PRESSURE	POWER	HEAD	WHP	EHP	e_w
(CFS)	(PSI)	(PSI)	(KW)	(FT)	(%)	(%)	(%)

(1) PUMP HEAD = 2.31 * (DOWNSTREAM PRESSURE (PSI) - UPSTREAM PRESSURE (PSI)) (2-4)

(2) WATER HORSEPOWER = FLOW (CFS) * PUMP HEAD (FT) * $\gamma/550$ (2-5)

(3) ELECTRICAL HORSEPOWER = POWER (kW)/0.746 (2-6)

(4) WIRE-TO-WATER EFFICIENCY = WHP/EHP (2-7)

Figure 2-1. Test Pump Data Sheet

7. Example Problem. Consider the pump system shown in Figure 2-2. From the measured quantities tabulated below, develop head versus discharge, water horsepower versus discharge, electrical horsepower versus discharge, and wire-to-water efficiency versus discharge curves. The pressure (p) readings were made at a pressure gage downstream of the pump (elevation 930 ft), while the discharge and power readings were being measured. The computation steps required for construction of the pump characteristic curves are shown below. The first data point from the measured quantities is used in the example computations. The results for the entire data set are tabulated as computed quantities. The resulting characteristic curves are shown in Figure 2-3.

Measured Quantities			Computed Quantities			
Q (gpm)	ΔP (psi)	Power (kw)	h_p (ft)	WHP	EHP	e_w
1,053	87	58	181	48	78	62
850	96	56	202	43	75	58
660	102	53	215	36	71	51
0	115	43	245	0	58	0

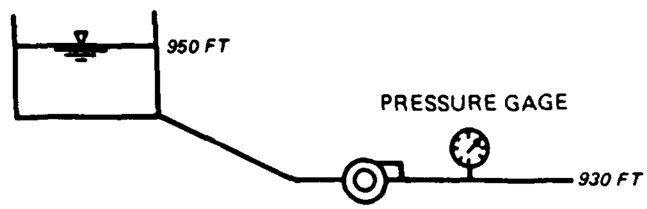

Figure 2-2. Example Pump System

a. Pump Head. The pump head is calculated by subtracting the head upstream of the pump from the downstream head:

$$h_p = H_d - H_u \tag{2-8}$$

(1) Upstream head.

H_u = 950 - losses in suction line

(For the example problem, suction line losses are negligible.)

(2) Downstream head.

$H_d = 930 + 2.31P + V^2/2g$ + losses between pump and gage

$V^2/2g = [Q/(\pi D^2/4)]^2/2g = [(Q/448)/(\pi * 1/4)]^2/64 = 1.2 \times 10^{-7}\ Q^2$

(Thus, the velocity head is negligible.)

(3) Pump head.

$h_p = 930 + 2.31P - 950 = 2.31P - 20$

For example, for Q = 1,053 gpm and P = 87 psi, h_p = 181 ft.

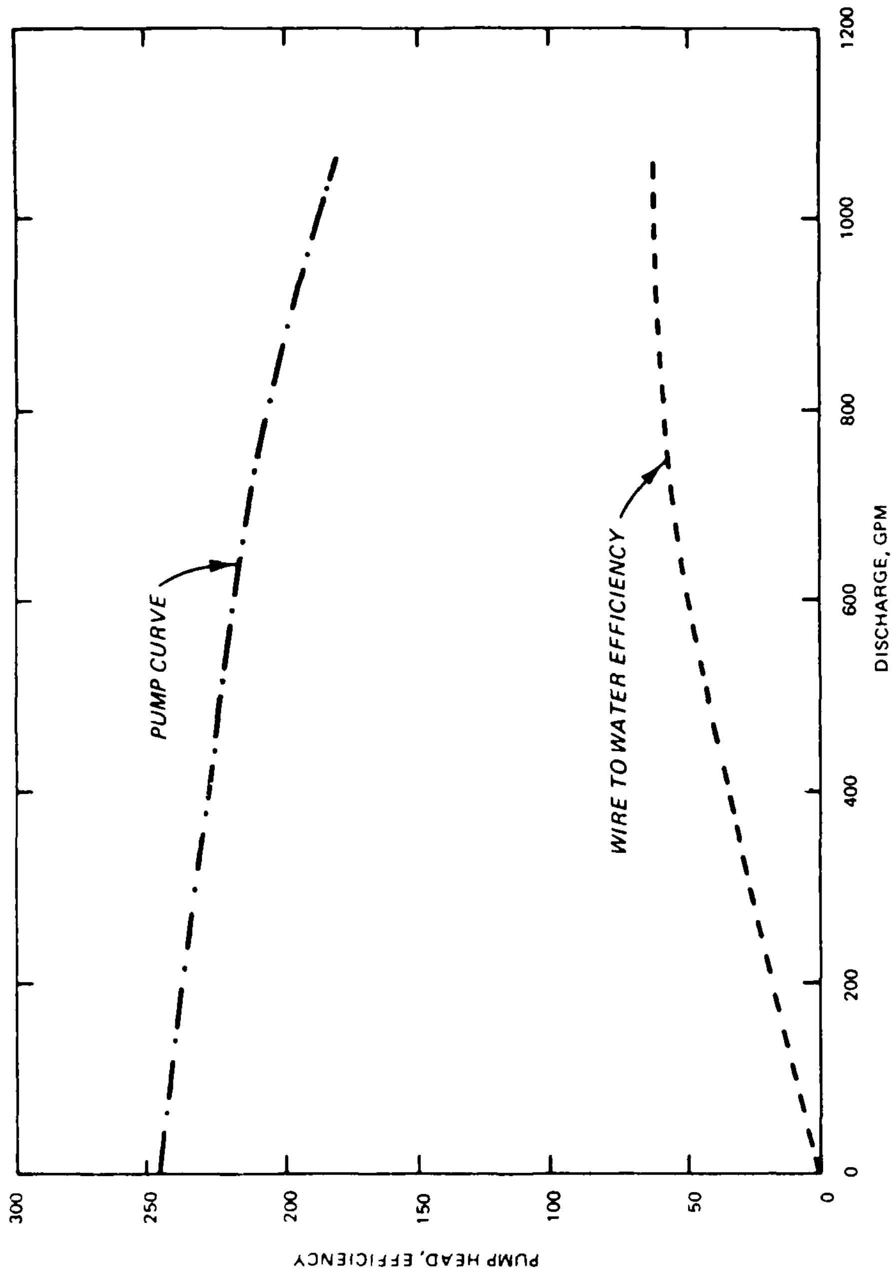

Figure 2-3. Pump Characteristic Curves

b. Water Horsepower.

$$WHP = \frac{(Q/448)\ h_p (62.4)}{550} = (0.000253)\ Q\ H$$

For example, for Q = 1,053 gpm and H = 181 ft, WHP = 48.3 hp.

c. Electrical Horsepower.

$$EHP = \frac{kW}{0.746} = 1.43\ kW$$

For example: for Q = 1,053 gpm and kW = 58, EHP = 77.7 hp.

d. Wire-to-Water Efficiency.

$$e_w = \frac{WHP}{EHP}$$

For example: for Q = 1,053 gpm, WHP = 48.3 hp, and EHP = 77.7 hp, e_w = 0.62.

Section II: Field Measurements

As indicated previously, the head-discharge and head-efficiency characteristics curve may be constructed from corresponding measurements of head, discharge, and power. Guidelines for measuring each of these quantities are provided in the following sections.

8. Head Measurement. The head values used in constructing the head-discharge curves correspond to the net head delivered by the pump. The net head (h_p) may be obtained by subtracting the head measured on the suction side of the pump (H_s) from the head measured on the discharge side of the pump (H_d) (see Figure 2-4). In equation form, this may be expressed as:

$$h_p = H_d - H_s \qquad (2\text{-}9)$$

where h_p = total head delivered by pump, ft

H_d = discharge head, ft

H_s = suction head, ft

Both the suction head and the discharge head are composed of three different components: the elevation head, the velocity head, and the pressure head. In general, the total head on either the suction side or the discharge side of the pump may be expressed as:

$$H = z + V^2/2g + P/\gamma \qquad (2\text{-}10)$$

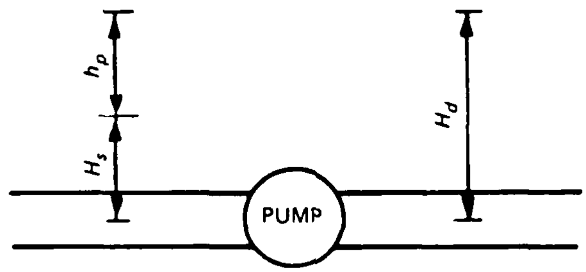

Figure 2-4. Pump Head Measurement

where z = elevation, ft

$V^2/2g$ = velocity head, ft

P/γ = pressure head, ft

V = velocity, ft/sec

P = pressure, psf

If the suction line and the discharge lines both have the same diameter and the elevations at which the pressures measurements are made are essentially the same, the net head will simply be the difference between the suction pressure and the discharge pressure expressed in feet:

$$h_p = 2.31\ (P_d - P_s) \tag{2-11}$$

where P_d = discharge pressure, psi

P_s = suction pressure, psi

a. Suction Pressure. Due to the low pressures normally associated with the suction line, compound (pressure/vacuum) gages, water columns, or manometers are commonly used to measure the suction pressures. When water columns are used, care should be taken to avoid errors due to the difference between the temperature of the water in the gage and that of the water in the pump (Hydraulic Institute 1983).

b. Approximate Suction Head. In installations where the water is being lifted out of a clearwell and a suction gage or manometer is not available, an approximate head can be obtained by measuring the vertical distance from the level of the water in the clearwell to the center of the discharge pressure

gage, adding an appropriate value for loss in bends, velocity head, etc. (Gros 1983).

c. Discharge Pressure. Due to the high pressures normally associated with the discharge line, mercury manometers, bourdon gages, and electrical pressure transducers are usually used to measure the discharge pressure. Before and after bourdon gages or pressure transducers are used, they should be calibrated.

d. Pressure Taps. In order to measure the head differential across a pump, the head-measuring instruments must be attached to both the suction and discharge ends of the pump. In most cases pressure taps are generally available for such purposes. For those installations where such taps are not available, they should be installed using the guidelines established by the Hydraulic Institute (1983).

e. Flow Requirements. It is important that steady flow conditions exist at the point of instrument connection. For this reason, it is generally necessary that the pressure or head measurement be taken on a section of pipe where the cross section is constant and straight. Five to ten diameters of straight pipe of unvarying cross section following any elbow or curved member, valve, or other obstruction are necessary to ensure steady flow conditions (Hydraulic Institute 1983).

f. Head Loss Requirements. If the pipe friction loss between the pump discharge flange and the point of instrument connection can be significant, it should be added to the total head. The friction factor used for the calculation should be based on the appropriate roughness for the actual pipe section. A potential problem in conducting field tests is the head loss through partially closed valves. As a result, pressure head measurements should be made between the pump and the valves where possible.

9. Flow Rate Measurement. Liquid meters may be classified into two different functional groups. One group primarily measures quantity while the other primarily measures rate of flow. Quantity meters include weighing meters and volumetric meters. Rate meters, include head (kinetic) meters, area meters, head-area meters, velocity meters, and additional special methods. General guidelines for the use of these meters may be obtained from the Hydraulic Institute (1983) Standards and Walski (1984b).

a. Kinetic Meters. Probably the most commonly used meters are the head (kinetic) meters. Meters falling into this classification include venturi meters, nozzle meters, and orifice plate meters. For meters of this type, the average discharge may be obtained using the following equation:

$$Q = C\sqrt{2g\,(\Delta h)} \qquad (2\text{-}12)$$

where Q = discharge, cfs

C = meter coefficient

Δh = head differential across meter, ft

g = gravitational constant (32), ft/sec^2

The discharge coefficient is available from the manufacturer. Measurement of differential head is described below.

b. Differential Head Measurement. Several types of devices are available for measurement of the differential head across the meter. These range from manometers in which the difference in pressure is balanced by different heights of fluid columns, to bellows-type differential pressure gages in which the change in position of the bellows is converted into a dial reading, to electronic pressure transducers where differential pressure is converted into an electrical signal. Manometers to be used with water can further be subdivided into heavy and light liquid manometers, depending on whether the manometer liquid is heavier or lighter than water, and air-filled manometers in which air is used as the manometer fluid. Differential pressures can also be obtained by measuring the pressure using two pressure gages and subtracting the readings. For water distribution systems, gage pressure is usually much higher than the differential pressures and, therefore, this two-gage approach is not sufficiently accurate. A discussion of the merits of each of the above approaches is provided by Walski (1984a).

10. Horsepower Measurement. The electrical horsepower required by the pump is determined by measuring the kilowatt demand directly or by calculating the kilowatt demand using measured values of voltage and current. For stations where the power demand for each pump has been instrumented, the readings may be obtained directly from the instrument panel. However, care should be taken to ensure that the instrumentation has been properly calibrated (Lackowitz and Petretti 1983).

a. Direct Measurement. For those stations where instrumentation is not provided, the power may be measured from the electric meter. Normally, the integrations on most electric meters in pumping stations cannot be read closely enough to be accurate. However, by counting the number of revolutions on the revolving disk, an estimate of the power uptake may be obtained using the following equation:

$$\text{Power (kW)} = 3.6 * (\text{drps}) * (\text{Kh factor}) \qquad (2\text{-}13)$$

where drps = disc revolutions per second

Kh factor = a disc constant

The Kh factor represents the hundredths of kilowatt hours per revolution and is usually stamped on the meter face. If the meter includes other power uses in addition to the pump, these uses must be subtracted. In this case, care must be exercised to ensure that other equipment does not kick on during the pump test (Lackowitz and Petretti 1983).

b. Example. If a watt-hour meter revolves at 12 rpm and the Kh factor for the meter is 52.5, determine the power usage.

$$\text{drps} = (12 \text{ rev/min}) * (1 \text{ min}/60 \text{ sec}) = 0.2 \text{ rps}$$

$$\text{Power (kW)} = 3.6 * 0.2 * 52.5 = 37.8 \text{ kW}$$

c. Indirect Measurement. Where standard kilowatt meters are not available, the power input can also be determined indirectly. This is done by determining the line current (amperes), the line-to-line voltages (volts), and the power factor on the load side of the motor-starting device by using a poly-phase watt meter or watt/VAR-power factor/volt-ampere (PF/VA) demand meter. These types of meters can be purchased or rented from several leading manufacturers of electrical instrumentation and measuring devices. Some of the meters are fairly sophisticated, incorporating digital or dial displays and data-recording features. In general, this approach is better than deriving the power consumption from a standard kilowatt meter device since the power factor for the prime mover can also be obtained. However, due to the danger involved, such tests should only be done by a skilled electrician (Lackowitz and Petretti 1983).

d. Example. If the line current is measured to be 200 amps and the voltage is measured to be 2.4 kV, determine the power demand in kW if the power factor for the motor is determined to be 0.70.

$$\begin{aligned} \text{Power (kW)} &= PF * I_{\lambda} * V_{\lambda} * \sqrt{3} \div 1000 \\ &= 0.70 * 200 \text{ amps} * 2{,}400 \text{ volts} * \sqrt{3} \div 1{,}000 \\ &= 582 \text{ kW} \end{aligned}$$

Section III: Pump Efficiency Evaluation

11. Region of Pump Operation. Once each pump has been field tested, both the head-discharge and the discharge-efficiency curves for each pump should be constructed using the test data. Once these curves have been developed they should be superimposed on the band of system head curves for the corresponding network as shown in Figure 2-5. The intersection of the band of system head curves with the pump head-discharge curve corresponds to the region of expected pump operation. The band of curves may be obtained by plotting the maximum and minimum system head curves as shown in the figure. The highest curve would correspond to the suction tank being nearly empty, the discharge tank being full, other pumps pumping into the downstream pipes, and most water users being located beyond the tank. The lowest curve would correspond to the opposite conditions (i.e., suction tank full, discharge tank nearly empty, no other pumps operating, and most water use occurring between the pump and the discharge tank). The point on the curve determined with none of the valves throttled should fall between the maximum and minimum system head curve unless unusual conditions existed on the system at the time of the testing.

12. Region of Efficient Operation. For efficient pump operation, the head versus discharge curve should intersect the band of system head curves in the region of maximum efficiency. Over a period of several years the actual operating point may change from the original design point due to wear on the pump or changes in the system. If the pump is not operating near the region of maximum efficiency, either the band of system head curves or the pump characteristic curve may be modified in order to shift the intersection point of the two curves to the desired region.

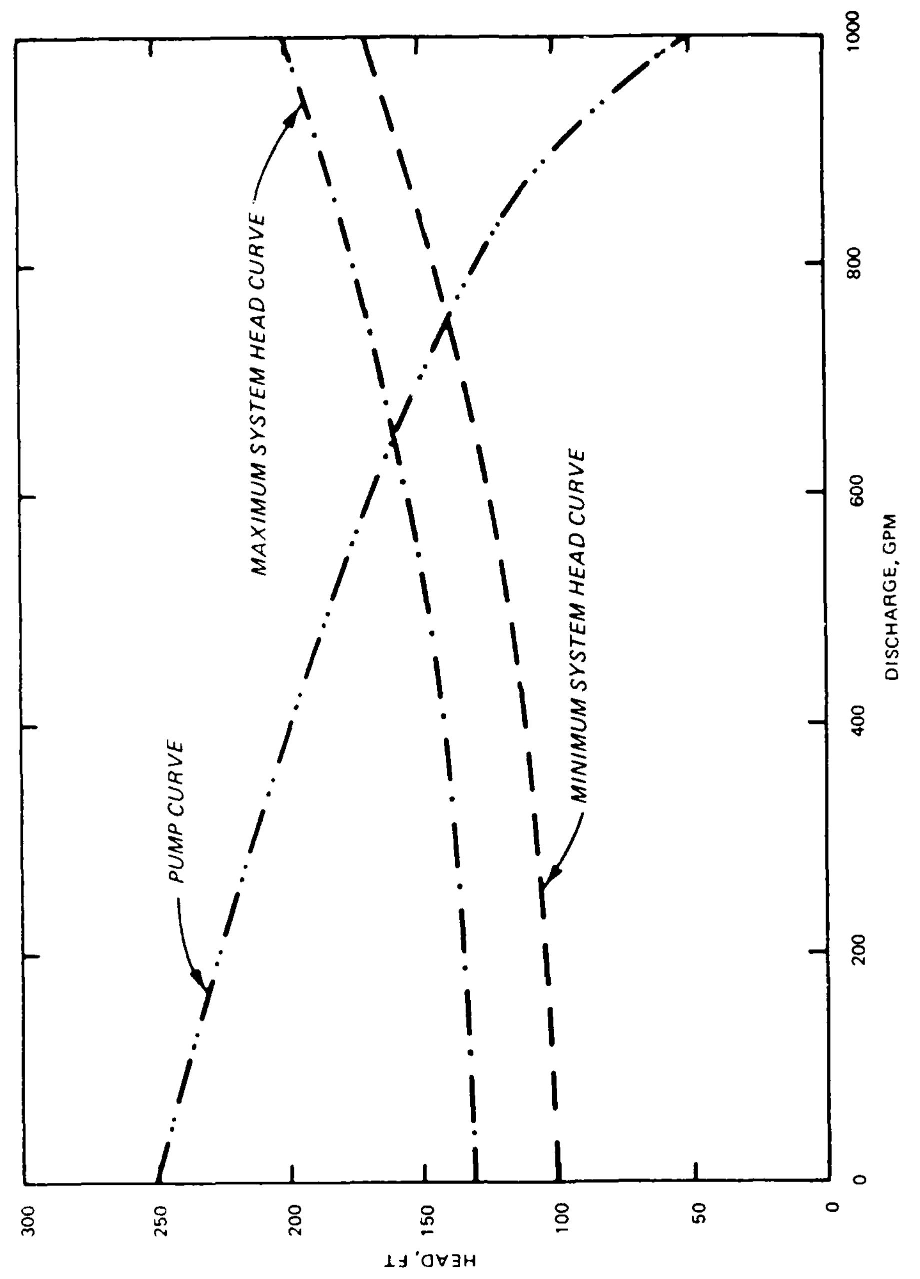

Figure 2-5. Pump Operating Range

13. Modification of the System Head Curve. If the actual operating point lies to the right of the desired region, the operating point may be shifted to the left by raising the system head curve. This may be accomplished by adding additional head loss to the system. Additional head loss may be added to the system head curve by a valve downstream of a pump. Although such a procedure will indeed result in additional head loss, in some cases it may actually result in lower energy costs due to improved efficiency. If the actual operating point lies to the left of the desired region, the operating point may be shifted to the right by lowering the system head curve. This may be accomplished by decreasing the head loss in the system. One possible way to decrease the head loss would be to add parallel lines to the main supply line or have the existing lines cleaned.

14. Modification of the Head Discharge Curve. The alternative to modifying the system head curve in order to improve the efficiency of the operating point is to modify the head discharge curve of the pump. The head discharge curve may be modified by either changing or modifying the existing pump impeller, or using a driver with a different speed.

15. References.

a. American Water Works Association (AWWA), 1977, American National Standards for Vertical Turbine Pumps, ANSI/AWWA E101-77.

b. Gros, W. F., 1983, "Wasting Energy? Use This Simple Method to Check Pump Efficiencies," Proceedings of the AWWA Summer Convention.

c. Hydraulic Institute, 1983, Hydraulic Institute Standards for Centrifugal, Rotary, and Reciprocating Pumps, Cleveland, Ohio.

d. Lackowitz, G. W., and Petretti, P. J., 1983, "Improving Energy Efficiency Through Computer Modeling," J. AWWA, Vol 25, No. 10 pp 510-515.

e. Walski, T. M., 1984a, "Application of Procedures for Testing and Evaluating Water Distribution Systems," Technical Report EL-85-5, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

f. Walski, T. M., 1984b, Analysis of Water Distribution Systems, Van Nostrand Reinhold, New York.

MULTIPLE PUMP OPERATION

1. Introduction. For multiple pump stations, potential energy savings can be identified by examination of the overall efficiencies associated with the operation of different combinations of pumps. Although different combinations of similar pumps may deliver the same approximate flow rate, some combination for each flow may be less costly because of differences in pump efficiencies. In some cases, the efficiency of a pump when running alone can be significantly different than when it runs in conjunction with other pumps.

Section I: Instantaneous Operation

2. Composite Pump Curves. In order to identify which combination of pumps is most efficient for a given flow rate, the composite characteristic curves for each combination of pumps must be superimposed on the system curve of the appropriate service area. The intersection of these two curves will indicate the combined operating point (head and combined flow) for the particular combination of pumps (Matsumoto and Mays 1979).

a. Parallel Pumps. The composite characteristic curve for a set of pumps in parallel can be obtained by adding the capacities of the two pumps at each head (see Figure 3-1). Discharge does not increase at heads above the maximum head of the smaller pump. In addition, a second pump will produce flow only when its discharge head is greater than the discharge head of the pump already running. When the parallel characteristic curve is plotted with the system head curve, the operating point is the intersection of the system head curve with the A+B curve. For parallel pumps, each pump will be pumping against the same head. The flow rate and efficiency for each pump can be obtained by referring to the pump characteristic curves for each individual pump and then reading off the flow rate and efficiency values corresponding to the total head.

b Series Pumps. The composite head-discharge curve for a set of pumps in series can be plotted by adding the heads of the two pumps at each flow rate (see Figure 3-2). When the series head-discharge curve is plotted with the system head curve, the operating point is again the intersection of the system head curve with the A+B curve. For series pumps, each pump will be pumping the same discharge; the head and efficiency for each pump can be obtained by referring to the pump characteristic curves for each individual pump and reading the head and efficiency values corresponding to the total discharge.

3. Energy Consumption for Pump Combinations. When more than two pumps are present, the same procedure discussed above can be applied to each possible pump combination. Once the combined operating point for each pump combination has been determined, the head, flow rate, and wire-to-water efficiency associated with each pump should be determined. Once the head, flow rate, and efficiency of each pump has been determined, the total required kilowatt power for the system of pumps can be determined using the following equation:

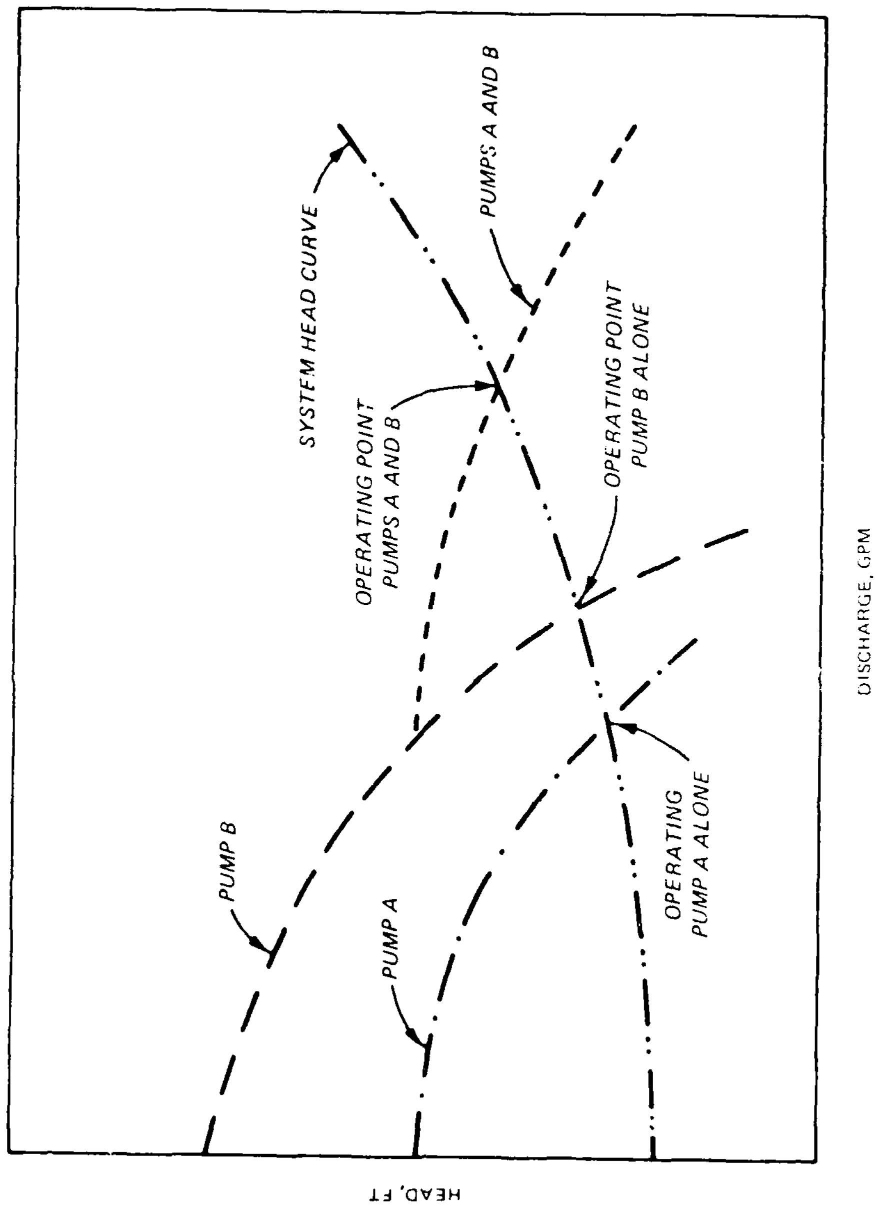

Figure 3-1. Parallel Pump Operation

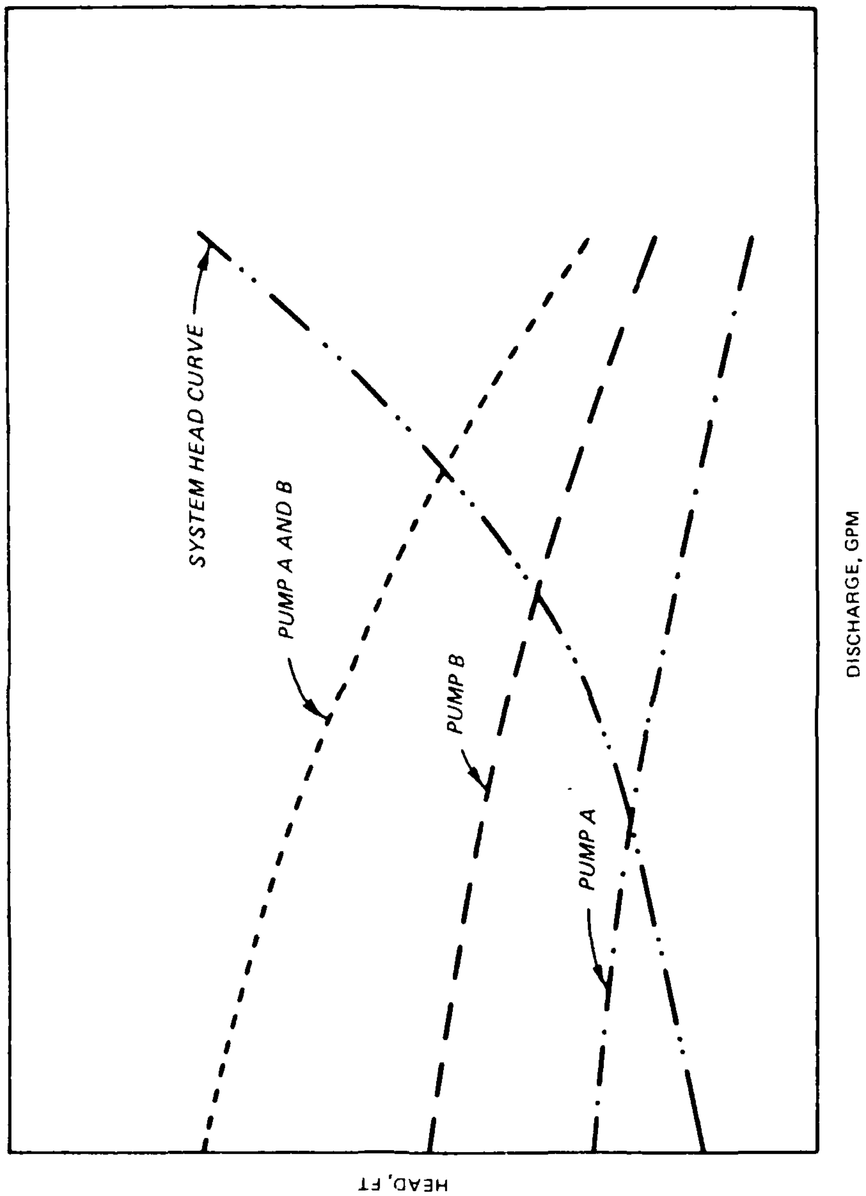

Figure 3-2. Series Pump Operation

$$kW \sum_{i=1}^{I} \frac{Q_i\, h\, \gamma}{550\, e_i} * 0.746 = \frac{h\, \gamma\, 0.746}{550} \sum_{i=1}^{I} \frac{Q_i}{e_i} \qquad (3-1)$$

where

kW = required power, kilowatts
I = total number of pumps
Q = pump discharge, cfs
h = pump head, ft
γ = specific weight of fluid, lb/ft^3
e = wire-to-water efficiency, fraction

For a given kilowatt demand charge and an energy price rate, the cost of each pump combination may be determined. Once the cost of each combination has been determined, each combination may be ranked from the most costly to the least costly. It is best to make cost comparisons on a cost per unit volume basis (e.g., cents per thousand gallons, kgal). Once this has been accomplished, the most efficient pump combination for a given flow rate can be determined.

4. Example. For the combined operating point (for two parallel pumps) shown in Figure 3-3, determine the total kilowatt demand and energy cost of 9 cents per kilowatt-hour.

a. Pump Head (from Figure 3-3).

Total Pump Head = 200 ft

b. Pump Discharge (from Figure 3-3).

Discharge for Pump A = 1,000 gpm = 2.23 cfs
Discharge for Pump B = 1,600 gpm = 3.57 cfs

c. Pump Efficiency (from Figure 3-4).

Efficiency for Pump A (Q = 1,000 gpm), e = 0.76
Efficiency for Pump B (Q = 1,600 gpm), e = 0.78

d. Kilowatt Demand.

$$kW = \frac{h\gamma\ 0.746}{550} \sum_{i=1}^{2} \frac{Q_i}{e_i} = \frac{9,310}{550} * \left[\left(\frac{2.23}{0.76} + \frac{3.57}{0.78}\right)\right] = 127 \text{ kW}$$

e. Cost.

(127)(0.09) = \$11.4/hr
\$100,127/yr

f. Unit Cost.

$$\frac{\$11.4}{hr}\left(\frac{min}{2,600\ gal}\right)\left(\frac{1,000\ gal}{kgal}\right)\left(\frac{hr}{60\ min}\right)\left(\frac{100¢}{\$}\right)$$
= 7.3 ¢/kgal

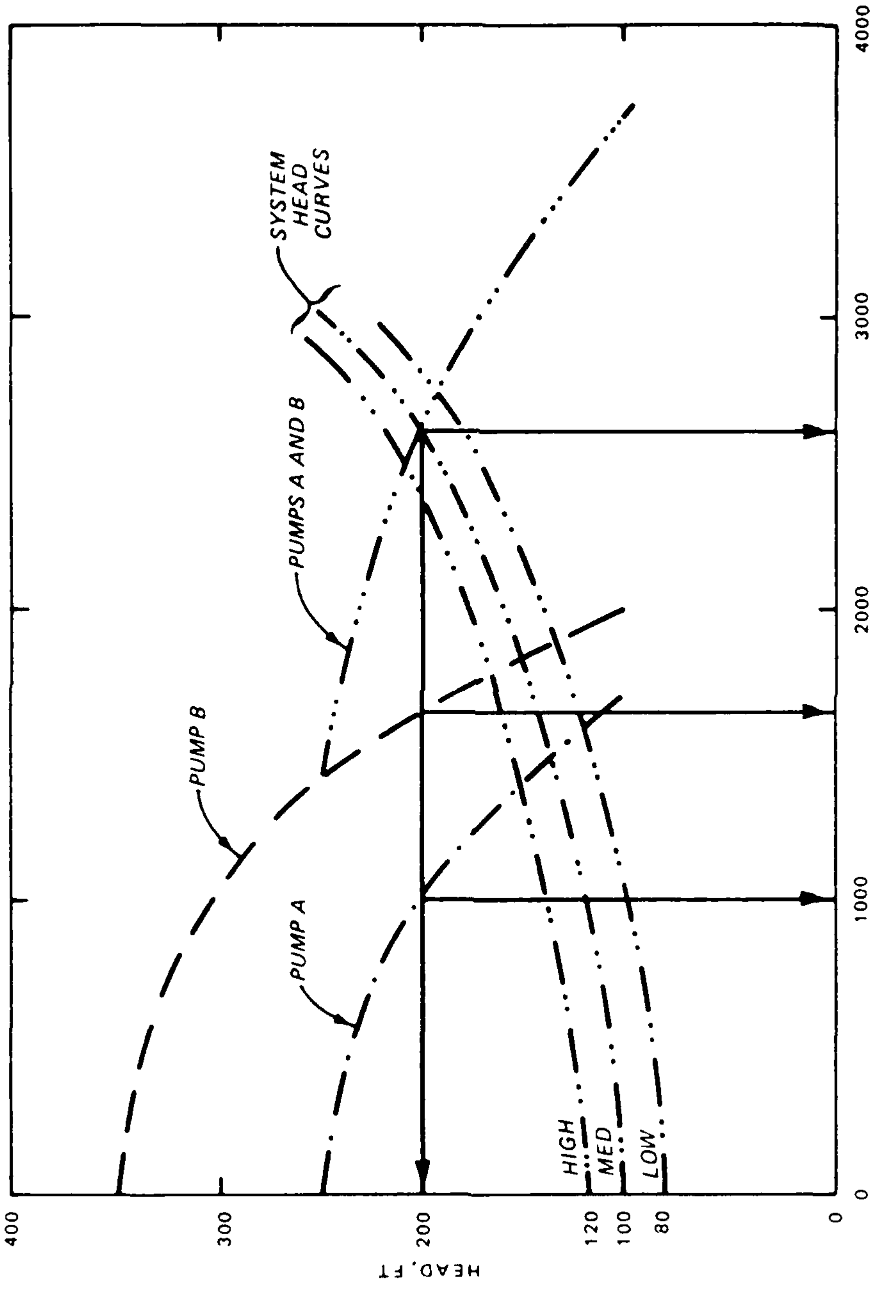

Figure 3-3. Example Head Versus Discharge Curves

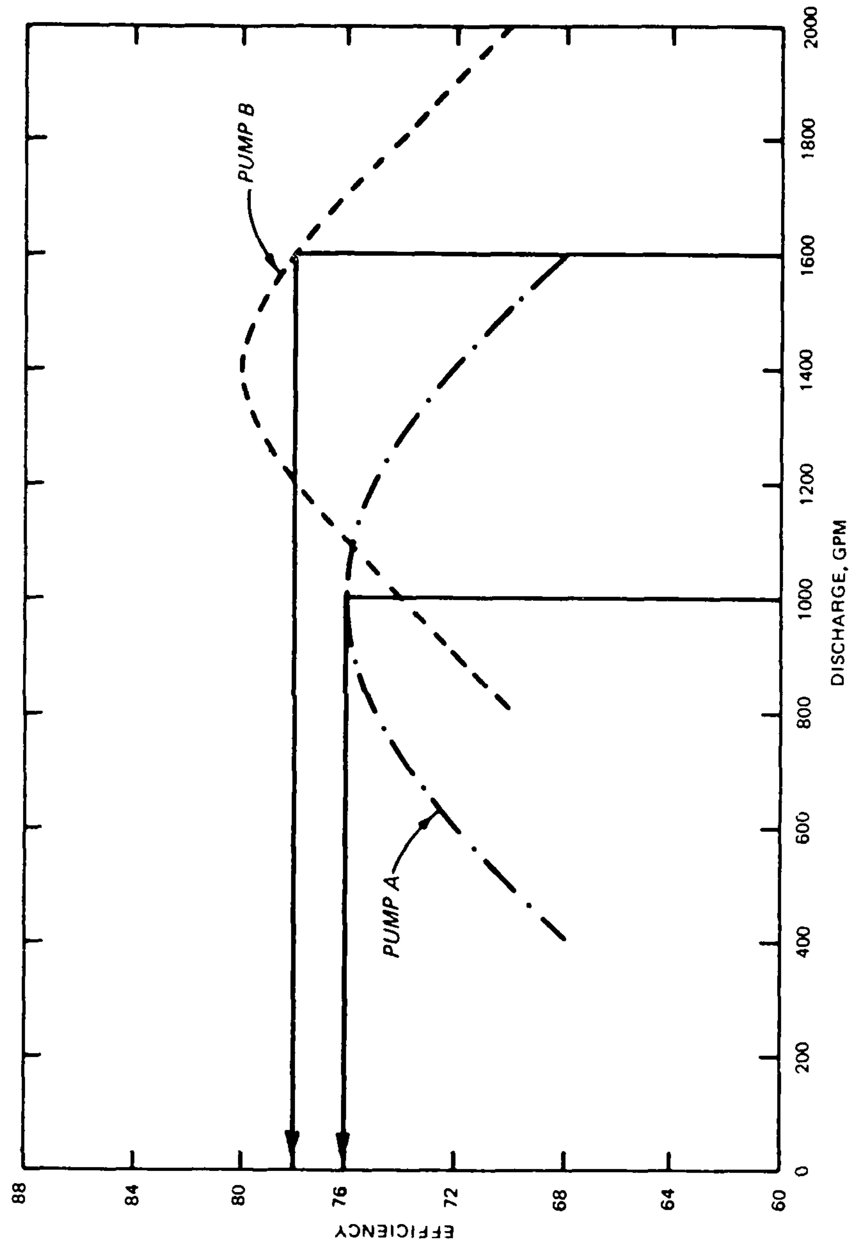

Figure 3-4. Example Efficiency Versus Discharge Curves

Table 3-1 shows these calculations for several tank levels (low, medium, and high) and each pump and combination. In general, it is less expensive to operate pump A than pump B, and pump A and B should only be operated together when demands are very high.

Table 3-1. Unit Cost for Example Problems

Pump Running	System Curve Head	h, ft	Q_i, cfs	e_i	Q_{total} cfs	kW	¢/kgal
A	LOW	123	3.49	0.68	3.49	54	5.1
B	LOW	137	4.22	0.72	4.22	68	5.4
A + B	LOW	193	2.38	0.76	6.09	130	7.2
			3.71	0.76			
A	MED	138	3.29	0.70	3.29	55	5.6
B	MED	152	4.03	0.74	4.03	70	5.8
A + B	MED	200	2.23	0.76	5.80	127	7.3
			3.57	0.78			
A	HIGH	152	3.10	0.72	3.10	55	6.0
B	HIGH	166	3.92	0.75	3.92	73	6.3
A + B	HIGH	207	2.02	0.76	5.56	126	7.6
			3.54	0.78			

Section II: Extended Period Pump Operation

5. Operating Policies. Although the previous procedures will allow the determination of the energy consumption and unit cost for a given set of conditions, the optimal combination may change as the system conditions change (e.g., tanks fill, demand varies). In practice, most pump operations are not changed continuously but kept constant over longer periods of time (e.g., several hours). For example, the operator can use a table such as Table 3-1 to select the least costly pump or combination to operate at any given time. The operator is not so much interested in selecting the optimal combination as he is in avoiding very inefficient combinations. However, in some cases, an operator may want to determine the optimal pump combination for the average set of conditions over a period of several hours. Such a procedure is presented below.

6. Pump Operation Graphs. For extended pump operations the optimal pump combination for a set of pumps may be determined graphically using a pair of simple-pump operation graphs. The first graph is called a static head-discharge graph and contains plots of discharge versus pump static head (not actual head) for an individual pump or combination of pumps. The second graph is called a static head-unit cost graph. This graph contains plots of static pump head versus the unit cost. The graphs are illustrated in Figure 3-5 for the simple case of a pump station with two different pumps. Static head is used in these graphs since it can be determine simply by subtracting the water

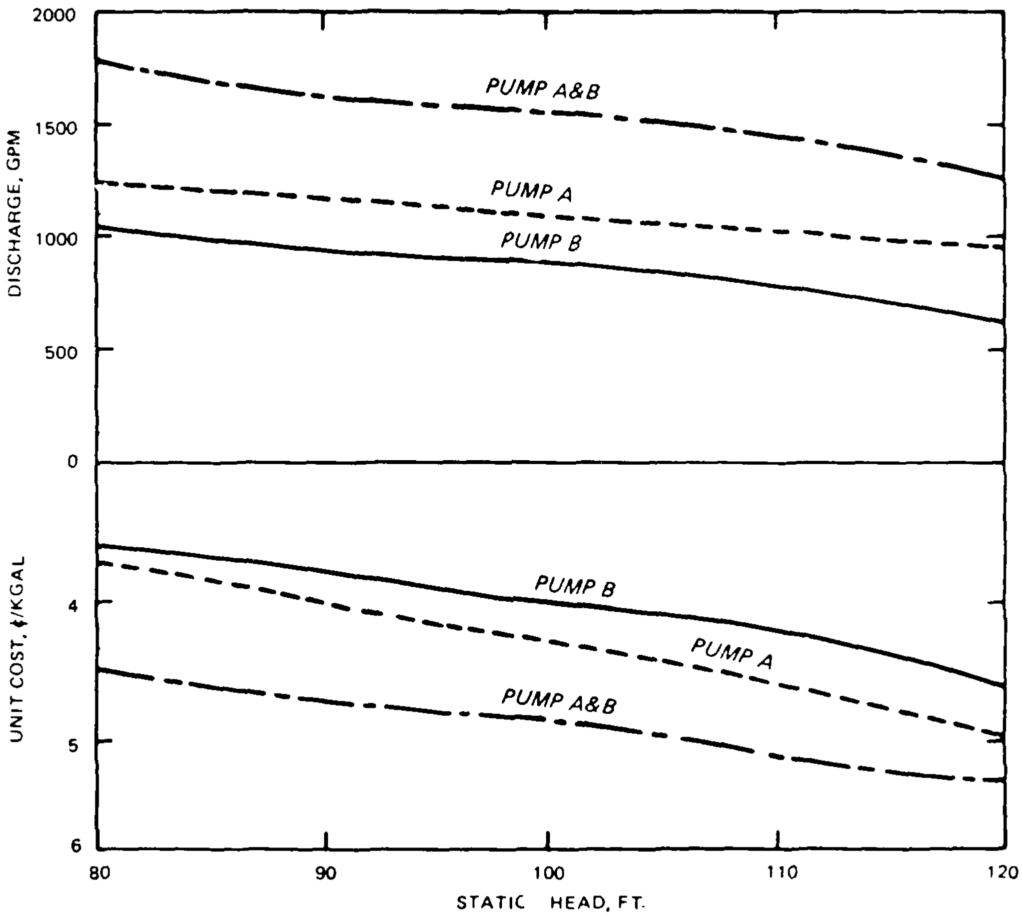

Figure 3-5. Static Head Versus Discharge and Unit Cost Graph

levels in the discharge and suction tanks. Unlike the pump head, it is independent of which pumps are operating.

a. Static Head-Discharge Graph. The static head versus discharge graph is constructed by superimposing the composite head versus discharge curve for each pump combination on the system head curve. Each curve can then be developed by plotting the flows associated with the combined operating points for different values of static pump head.

b. Static Head-Unit Cost Graph. The static head versus unit cost graph is constructed by plotting the unit cost associated with each discharge and pump combination versus the corresponding static pump head. The unit cost for a single pump can be obtained using the following equation:

$$\text{Unit Cost } (¢/\text{kgal}) = 0.00312hP/e_i \tag{3-2}$$

where

h = pump head, ft
P = price of electricity, ¢/kWhr

The unit cost for a multiple-pump combination may be obtained using the following equation:

$$\text{Unit Cost } (¢/\text{kgal}) = 0.00312hP \frac{\sum Q_i / e_i}{\sum Q_i} \tag{3-3}$$

where

Q_i = flow rate associated with pump i, gpm
e_i = overall efficiency associated with pump i

7. Example. Using the head versus discharge curves and the discharge versus efficiency curves for the two parallel pumps (pumps A and B) shown in Figures 3-6 and 3-7, generate static head versus discharge and static head versus efficiency curves for the band of system-head curves (static heads ranging from 80 to 120 ft) shown in Figure 3-6. In generating the unit cost curves, assume a power cost of $0.10/kWhr.

a. Pump A. Using the pump head characteristic curves for pump A and the discharge-efficiency curves shown in Figures 3-6 and 3-7 for pump A, construct the following table:

Static Head (ft)	Flow (gpm)	Head (ft)	Efficiency (%)	Unit Cost (¢/kgal)
80	1270	97	0.80	3.76
90	1180	106	0.81	4.06
100	1100	114	0.82	4.31
110	1020	122	0.83	4.56
120	940	128	0.82	4.98

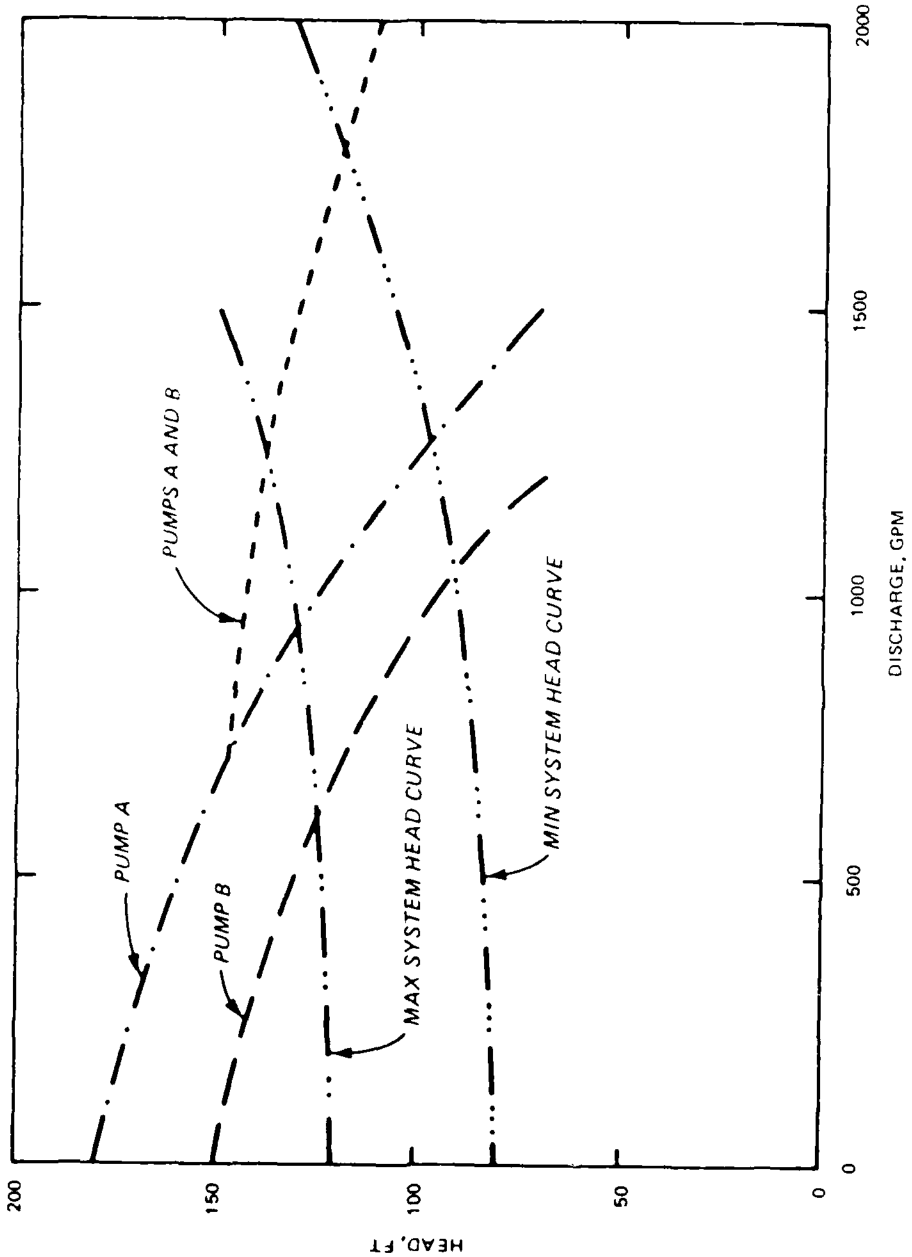

Figure 3-6. Example Head Versus Discharge Curves

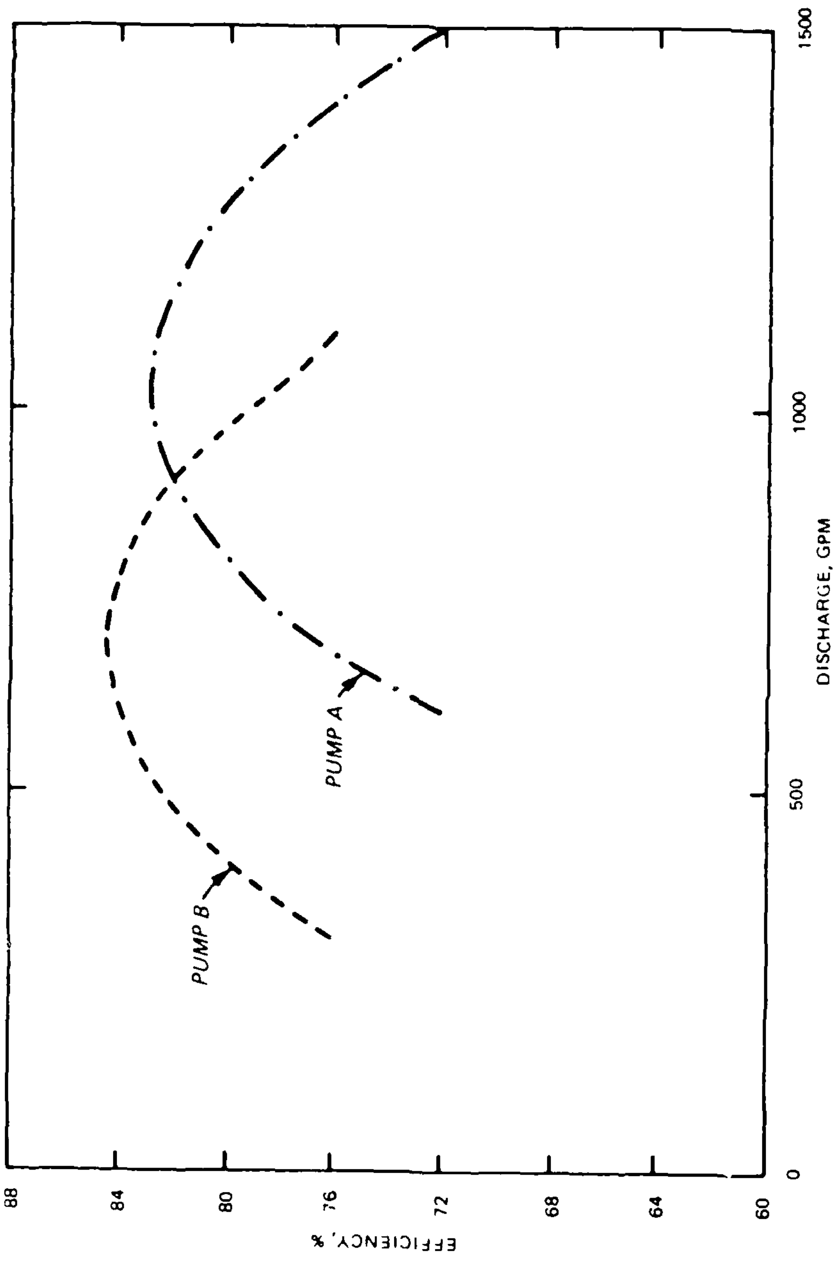

Figure 3-7. Example Efficiency Versus Discharge Curves

Once the table has been constructed, the relationship between static head and discharge (static head versus discharge graph) and the relationship between static head and unit cost (static head versus unit cost graph) can be plotted on Figure 3-5.

b. Pump B. Using the pump head characteristic curves for pump B and the discharge-efficiency curves shown in Figures 3-6 and 3-7 construct the following table:

Static Head (ft)	Flow (gpm)	Head (ft)	Efficiency (ft)	Unit Cost (¢/kgal)
80	1060	91	0.77	3.66
90	940	100	0.81	3.83
100	860	108	0.83	4.03
110	770	114	0.84	4.21
120	610	124	0.84	4.58

Once the table has been constructed, the relationship between static head and discharge and the relationship between static head and unit cost can be plotted on Figure 3-5.

c. Pumps A and B. Using the combined head versus discharge curve for pump A + B and the system head curves shown in Figure 3-6, the combined operating point for each static head can be obtained. For each operating point, the resulting total pump head can be obtained. Once this head has been obtained, it may be used with the individual head versus discharge and discharge versus efficiency curves of each pump to determine the corresponding discharges and efficiencies associated with each static head value. The unit cost values for the combined case may be obtained using Equation 3-3. The resulting values are tabulated below.

Static Head (ft)	Head (ft)	Total Flow (gpm)	Flow Pump A (gpm)	Eff. (%)	Flow Pump B (gpm)	Eff. Pump B (%)	Unit Cost (¢/kgal)
80	119	1,770	1,050	0.83	720	0.84	4.44
90	124	1,630	1,000	0.83	630	0.84	4.63
100	128	1,520	960	0.82	560	0.84	4.82
110	134	1,380	900	0.82	480	0.82	5.09
120	138	1,250	880	0.82	370	0.78	5.32

Once the relationship between the static head and discharge and the static head and unit cost have been determined, these values can be plotted on Figure 3-5.

8. Application of Pump Operation Graphs. Once the static head-discharge and the static head-unit cost graphs have been developed, they can be used to determine the most efficient pump combination for a set of specified operating conditions over a specified period of time. The procedure for using the graphs to obtain an optimal pump combination is summarized below.

a. Step 1. Starting with an initial static head (i.e., the difference in water surface elevation between the pump clearwell and the controlling elevated storage tank), determine the desired static head at the end of the desired operating period (e.g., 3 hours). The desired static head can be obtained by subtracting the projected clearwell surface elevation from the desired surface level in the elevated storage tank. Once both static heads have been determined, calculate the average static head for the specified operating period using the following equation:

$$h_{avg} = (Hdis_i - Hsuc_i + Hdis_f - Hsuc_f) / 2.0 \qquad (3\text{-}4)$$

where

h_{avg} = average static head, ft

$Hdis_i$ = initial head in discharge tank, ft

$Hsuc_i$ = initial head in suction tank, ft

$Hdis_f$ = final head in discharge tank, ft

$Hsuc_f$ = final head in suction tank, ft

If all heads remain constant during the specified operation period, then:

$$h_{avg} = Hdis - Hsuc \qquad (3\text{-}5)$$

b. Step 2. Next, determine the total volume of water expected to be demanded during the specified operating period. This requires knowing the use rate as a function of time of day, day of week, season, etc. The total amount of water that must be supplied during the specified operating period can be obtained from the following equation:

$$V_s = V_d + \Sigma \left[A_{tank} * (Hdis_f - Hdis_i) \right] \qquad (3\text{-}6)$$

where

V_s = total volume to be supplied, ft^3

V_d = total volume demanded, ft^3
= $8.02 * \Delta t$ (hr) * Q (gpm)

A_{tank} = average cross-sectional area of tank, sq ft

$Hdis_f$ = desired elevation of tank at end of period, ft

$Hdis_i$ = elevation of tank at the beginning of the period, ft

c. Step 3. Once the total required volume of water to be supplied has been determined, calculate the average flow rate required to deliver this volume using the following equation:

$$Q_{req} = 0.125 * V_s/T_o \qquad (3\text{-}7)$$

where

Q_{req} = the required average flow rate, gpm

V_s = total volume to be supplied, cu ft

T_o = time period, hr

d. Step 4. Once the average static head and the average flow rate have been determined, enter the static head versus discharge graph and draw a vertical line through the various curves corresponding to the average static head for the specified operating period. Now mark the points of intersection of this line and each of the pump combination curves. Each of these points represents the average operating point of the particular pump combination associated with the average static head. Next, mark a point on this line corresponding to the required average flow rate (see Figure 3-8).

Normally the required operating point will not correspond to one of the actual operating points for a particular pump combination. Instead, some of the points will be above the required point and some of the points will be below. The desired flow rate may be supplied using a combination of a point above and a point below (where each point will correspond to either an individual pump or a combination of pumps). The intersection of the static head line with the vertical axis of the graph (corresponding to zero flow) may also be considered one of the possible points below the operating point.

e. Step 5. Once the required operating point has been plotted, next determine all the feasible combinations of operating points between a point above and a point below the required operating point (see Figure 3-9). Next, determine the flow rates associated with each point. These flow rates may be determined directly from the vertical axis of the graph (see Figure 3-10). For each feasible combination of a point above and a point below the desired operating point, determine the percent of time required for each point in order to satisfy the required flow rate (see Figure 3-11). These percentages can be obtained by solving the following equations for f_a (the fraction of time required for the pump combination above the desired operating point) and f_b (the fraction of time required for the pump combination below the desired operating point).

$$Q_{req} = Q_b * f_b + Q_a * f_a \qquad (3\text{-}8)$$

where

$$f_b = (Q_{req} - Q_b)/(Q_a - Q_b)$$
$$f_a = (1 - f_b)$$

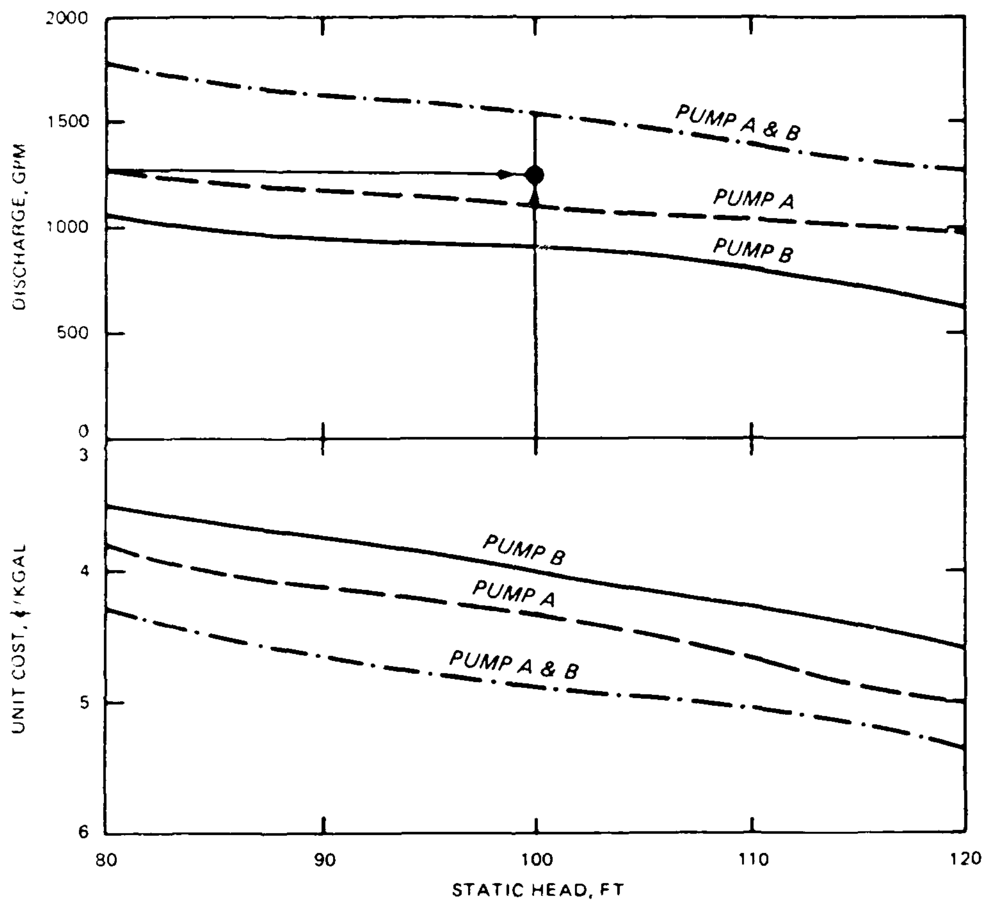

Figure 3-8. Determination of Operating Points

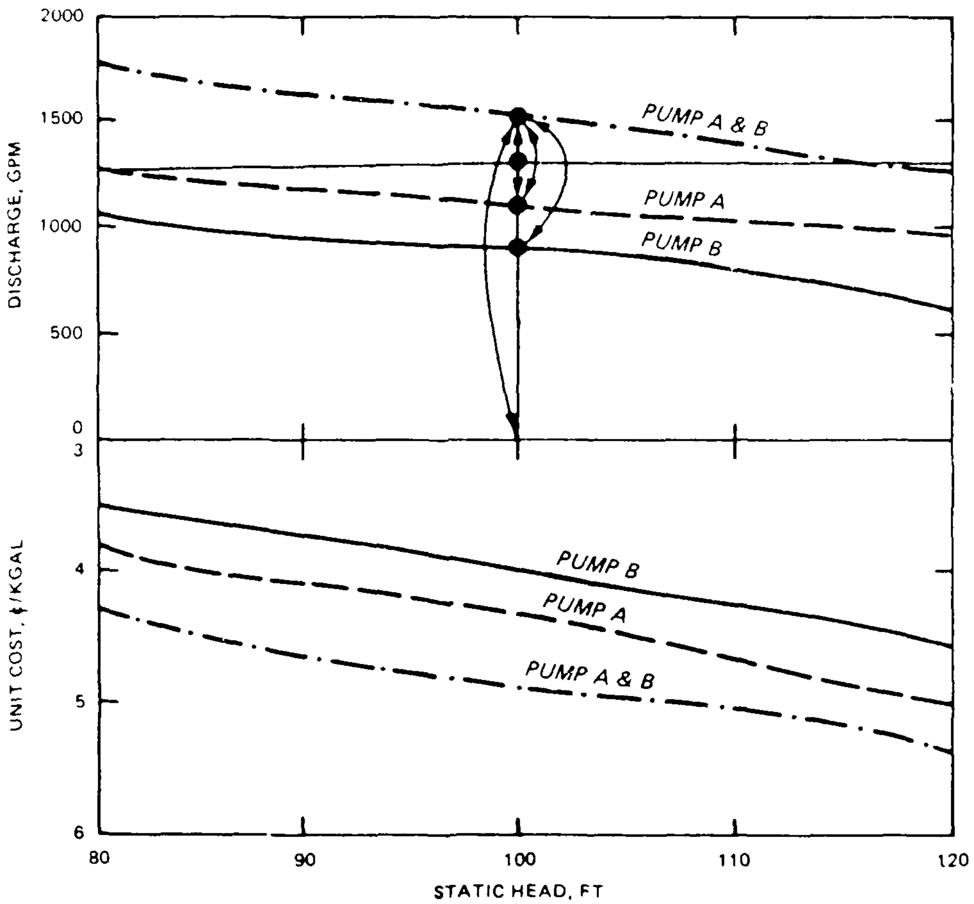

Figure 3-9. Feasible Pump Combinations

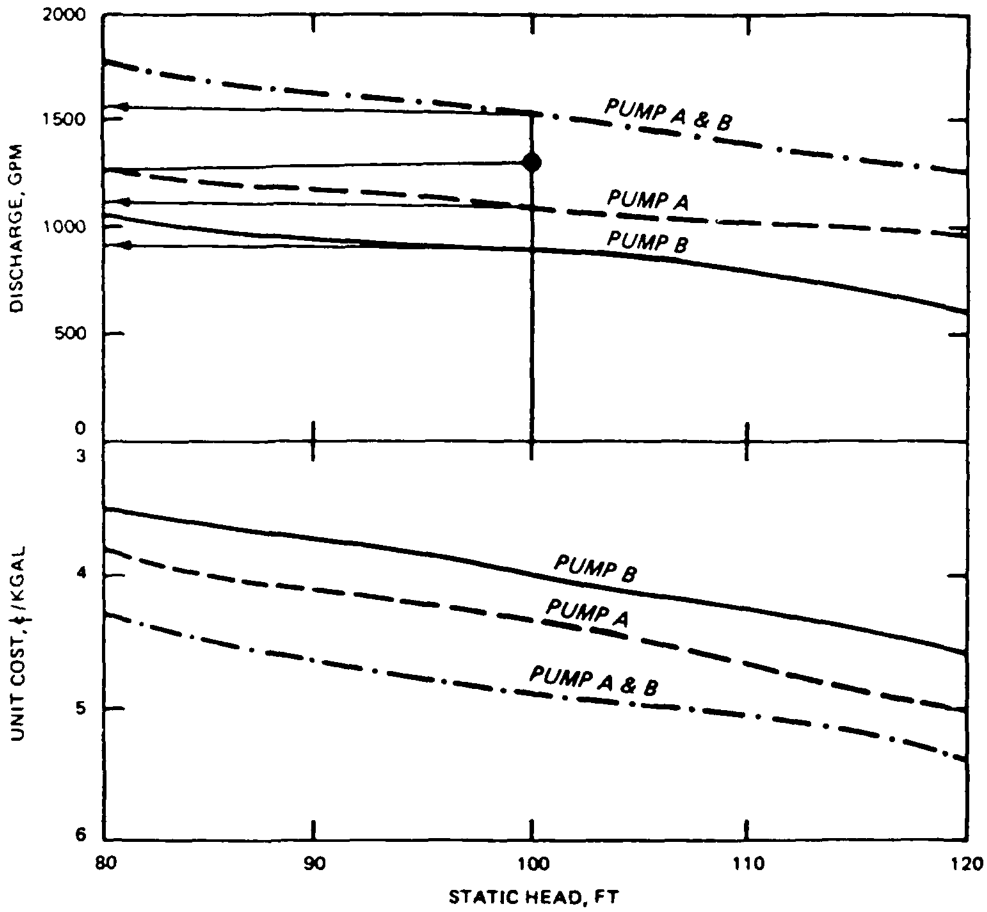

Figure 3-10. Flow Rates Associated with Pump Combinations

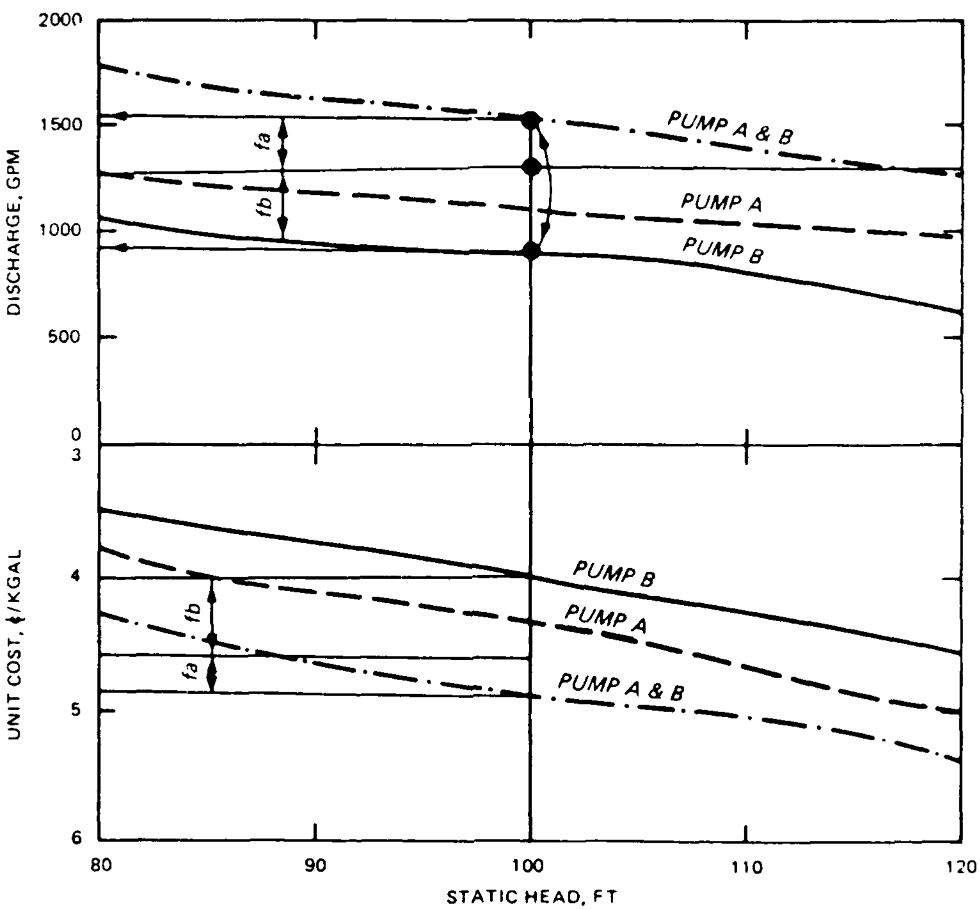

Figure 3-11. Percent of Time Associated with Pump Combinations

f. <u>Step 6</u>. Using the same combination of points selected in step 5 determine the unit costs associated with the flows for each of the operating points for each combination. These costs may be read directly from the static head-unit graphs as shown in Figure 3-11.

g. <u>Step 7</u>. Using the unit costs obtained from step 6, determine the total operating cost for each of the different operating combinations using the following equation:

$$C_i = 0.0006\ T_o(\ f_a Q_a C_a + f_b Q_b C_b) \qquad (3\text{-}9)$$

where

C_i = cost of operating combination i

T_o = total operating period, hours

C_a = cost of operating combination above Q_{req}

C_b = cost of operating combination below Q_{req}

h. <u>Step 8</u>. After the cost for each operating combination has been obtained, the costs can be ranked and the most cost-effective combination selected.

9. <u>Example</u>. During the next 4 hr, an operator wishes to raise the water level in an elevated tank from 479 ft to 481 ft while the clearwell water level is constant at 370 ft. The average cross-sectional area of the tank is 7,854 sq ft (50-ft radius) and the water use is 600 gpm during this time. Using the average static head versus discharge and average static head versus unit cost curves developed in the previous example, determine the optimal pump combination that will meet the required conditions.

a. <u>Step 1</u>.

$$h_{avg} = \frac{Hdis_f - Hsuc_f + Hdis_i - Hsuc_i}{2.0} = \frac{481 - 370 + 479 - 370}{2} = 110 \text{ ft}$$

b. <u>Step 2</u>.

$$V_d = 600 \text{ gpm} * 60 \text{ m/hr} * 4 \text{ hr} * 0.1337 \text{ cu ft/gal} = 19{,}250 \text{ cu ft}$$

$$V_s = V_d + A_{tank} * (Hdis_f - Hdis_i) = 19{,}250 \text{ cu ft} + 7{,}854 \text{ sq ft} * (481 - 479) = 34{,}960 \text{ cu ft}$$

c. <u>Step 3</u>.

$$Q_{req} = 0.125 * V_s/T_o = 0.125 * 34{,}960 \text{ cu ft}/4 \text{ hr} = 1{,}092 \text{ gpm}$$

d. <u>Step 4</u>.

See Figures 3-9 and 3-10.

e. Steps 5-8. Referring to Figures 3-9 and 3-10, three different pump operation combinations can be identified: Pumps A + B, Pump A and Pumps A + B, and Pump B and Pump A + B. For each combination, a unit cost must be determined:

(1) Pumps A + B.

Q_b = 0 (No pump on) | Q_a = 1,380 (Pumps A + B)

$$f_b = \frac{1,092 - 1,380}{0 - 1,380} = 0.20 \qquad f_a = 0.80$$

C_b = 0 ¢/kgal | C_a = 5.09 ¢/kgal

Cost = 0.0006 (4) 0.2 (0) (0) + 0.8 (1,380) (5.09) = $13.49

(2) Pump A and Pumps A + B.

Q_b = 1,020 (Pump A) | Q_a = 1,380 (Pumps A + B)

$$f_b = \frac{1,092 - 1,380}{1,020 - 1,380} = 0.80 \qquad f_a = 0.20$$

C_b = 4.56 ¢/kgal | C_a = 5.09 ¢/kgal

Cost = 0.0006 (4) 0.8 (1,020) (4.56) + 0.2 (1,380) (5.09) = $12.30

(3) Pump B and Pumps A + B.

Q_b = 770 gpm (Pump B) | Q_a = 1,380 gpm (Pump A + B)

$$f_b = \frac{1,092 - 1,380}{770 - 1,380} = 0.47 \qquad f_a = 0.53$$

C_b = 4.21 ¢/kgal | C_a = 5.09 ¢/kgal

Cost = 0.0006 (4) 0.47 (770) (4.21) + 0.53 (1,380) (5.09) = $12.59

(4) Based on the costs of each of the three possible operating decisions, the optimal decision is the second one (e.g., run pump A for 4 hr and pump B for 0.8 hr).

Section III: Computer Program

10. For pump systems with one or two pumps, the static head-discharge and static head-unit cost graphs can be generated fairly easily without the aid of a computer. The various operating combinations may also be evaluated fairly quickly. However, the number of pumps increases the construction of the graphs, and the evaluation of the various combinations can become somewhat tedious and time consuming. In order to facilitate the construction of the graphs or determine the optimal operating combinations, a computer program has been developed. A complete description of the computer program is provided by Ormsbee and Walski (1986).

11. References.

a. Matsumoto, J., and L. W. Mays, 1979, "Computerized Pump Analysis for Water Systems," ASCE Journal of the Environmental Engineering Division, Vol 105, No. EE1, pp 155-160.

b. Ormsbee, L. E., and T. M. Walski, 1986, Optimization of Water Supply Pumping Systems (Draft Report), Environmental Laboratory, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

PUMP OPERATION SCHEDULING

1. Introduction. Although the total energy consumption charges associated with a pump operation can be decreased by improving the efficiency of individual pumps or combinations of pumps, such measures have little impact on reducing the costs associated with time of day energy rate schedules. The primary way to minimize the cost associated with variable electric rate schedules is through the use of off-peak pumping strategies. The idea behind off-peak pumping strategies is to pump the water required to satisfy the peak system demands into elevated storage tanks during periods of slack demand (when electrical rates are normally lower). The excess water pumped during the slack demand period is used during periods of peak system demand (when the electrical rates are normally higher). The overall result of such a policy should be a lower total energy cost. It should be implemented if energy savings exceed the cost of additional storage.

2. Pump Operating Policies. The key to the implementation of an off-peak pumping strategy is the availability of equalizing storage and the development of an optimal pump operating policy. A pump operating policy is simply a schedule of water levels that should be maintained and a series of rules that dictate when different pumps should be operated in response to different system conditions. An optimal pump operating policy is that policy which will satisfy all system constraints at a minimum cost. Any optimal pump operating policy should be flexible enough to adjust to temporary changes in the normal operating conditions (e.g., fire events, pipe breaks, etc.).

Section I: Optimal Pump Operating Policies

3. Background. Although several authors have investigated the potential savings that may result from modification of an existing pump operation policies, very few authors have attempted to develop specific algorithms for use in improving such schedules. Due to the complexity and dynamic nature of many water distribution systems, it is difficult, if not impossible, to develop a least-cost operations procedure without the use of some type of optimization algorithm. Sterling and Coulbeck (1975a,b) have investigated using both heirarchial methods and dynamic programming in developing optimal pump operating policies, both with limited success. More recently, Sabet and Helwey ([illegible]) developed a dynamic program for use in an optimal pumping schedule for municipalities utilizing ground water as their water supply source. The algorithm was used to select the hourly operating schedule for three pumps for a system of 18 pipes with a single storage reservoir. For the example application, energy savings of approximately 35 percent were achieved. Unfortunately, the optimal solution did not require the initial and final states to be equal. In addition, the procedure neglects the impact of changes in system demand and pump combination on the system head curves.

4. Dynamic Programming Approach. In the current study an operations research technique known as dynamic programming is used to develop an optimal pump operating policy. Dynamic programming is simply a technique that is used to break large complex problems into a series of much smaller and simpler

ENCLOSURE 4

problems. The optimal solution to the larger problem is then found by summing the optimal solutions of the smaller subproblems. For problems that meet the requirements of dynamic programming, the optimal solution of the larger problem will be equal to the solution found by summing the optimal solutions of the smaller subproblems. For example, the solution of the daily or weekly pump operation problem may be determined by solving for the minimum energy costs over a small period of time (e.g., 1 hour) and then summing the costs over the entire period.

5. Problem Disaggregation. For the optimal pump operating policy problem, the "large" problem corresponds to determining the desired average static head (and corresponding tank water level) for each time interval (i.e., 1 hour) for a specified operating period (usually a day). The "smaller" subproblem corresponds to determining the minimum cost required to obtain a desired average static head for each time interval. Procedures for determining the optimal solution to the smaller subproblem (which pumps to run and at what cost) are presented in Enclosure 3. Procedures for solving the larger problem are presented in the following sections.

Section II: Problem Formulation

6. Preliminary Steps. Before the optimal pump operating policy problem can be solved, several preliminary steps must be taken. These are summarized below:

a. Determination of Problem Stages. Divide the desired operational period into discrete time intervals (usually hours). In dynamic programming terminology, the time intervals are known as "stages." The time intervals (or stages) are usually selected to correspond to changes in the electric rate or the system demand. After the operational period has been broken into different stages, the system demand and electric rate associated with each stage should be determined.

b. Determination of Problem States. Next, determine the maximum and minimum allowable static head (i.e., difference between suction tank (clearwell) water level and elevated storage tank water levels) values for each time interval. Once the maximum and minimum values have been determined, the range of values should be divided into reasonable intervals. Normally, a constant interval is used (usually several feet). For example, if water treatment plant clearwell level can fluctuate between 110 and 120 ft and elevated storage water level can fluctuate between 230 and 270 ft, the static head would range between 110 (230 - 120) and 160 ft (270 - 110) and states would be 110, 115, 120...160 ft if a 5-ft interval is used. In dynamic programming terminology, the variable associated with the different values of the static head is called a "state variable." In this case the state variable is simply the static head. For each stage (time interval), the state variable (static head) may be discretized into severable possible values. In dynamic programming terminology each possible value is called a "state." In mathematical notation an individual value of a state variable may be expressed as S_{ij} where i is a stage index and j is a state index. For example, if the static head is the state variable and the stages are 3 hr long, $S_{4,3}$ is the third possible state (e.g., 120 ft) with stage 4 (i.e., hour 12).

c. Construction of the State Space. The larger overall problem associated with determining the optimal pump operating policy can be visualized graphically through the construction of a "state-space diagram." The state-space diagram is simply a figure which shows the various states (static heads) that can occur at the beginning and end of a particular stage (time interval). A potential state-space diagram for a problem involving three different static heads (states) and four different time intervals (stages) is illustrated in Figure 4-1. Each state S_{ij} is represented by a circle, and each stage is represented by a square. In this example, a time interval of 6 hr is used. The values of the static heads (states) are assumed to be equal to 190, 200, and 210 ft. It should be noted that all the initial states are assigned a stage index of zero even though they represent the initial state of stage 1.

d. Construction of Transition Cost Matrices. The final step is to construct a transition cost matrix for each stage. This matrix shows the cost to move from a given state in one stage to another state in the next stage. An example transition cost matrix for the first stage of the example state space is shown in Figures 4-2 and 4-3. (Costs of infinity represent hydraulically infeasible transitions.) The transition cost matrix illustrates the cost of the pumping during stage (time interval) i required to change the tank level from its initial state $S_{i-1,j}$ to its final state $S_{i,k}$. In mathematical notation the cost associated with each transition may be expressed as C_{ijk}. The variable C_{ijk} represents the cost associated with the decision to change from state j at the beginning of stage i to state k at the end of stage i (i.e., the pumping cost required to go from $S_{i-1,j}$ to S_{ij}). For this problem the cost associated with each particular transition (the costs in each cell of the matrix) can be obtained using the procedures outlined in Enclosure 3 or some other rule. In this case, the average static head to be used with the operation curves is obtained as follows:

$$\bar{H}_{ijk} = \frac{S_{i-1,j} + S_{ik}}{2} \tag{4-1}$$

where $\bar{H}_{ijk}$ = the average static head

i = stage index associated with the final state

j = state at the beginning of stage i

k = state at the end of state i

The required volume to be used with the operation curves can be obtained in a similar manner as follows:

$$V_{ijk} = V_d + A_{tank} * (S_{i-1,j} - S_{ik}) \tag{4-2}$$

where V_d = volume required due to system demand

A_{tank} = average cross-sectional area of storage tank

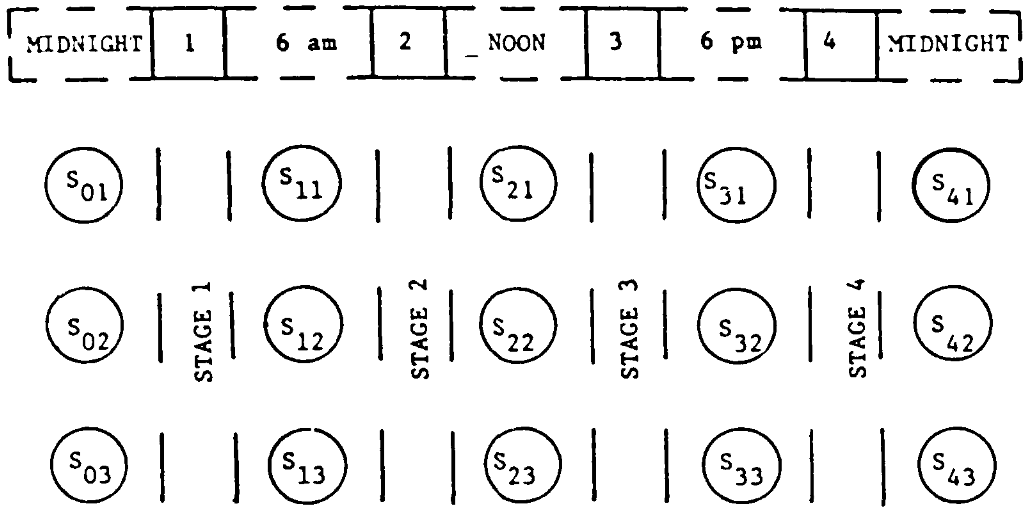

Figure 4-1. State-Space Diagram

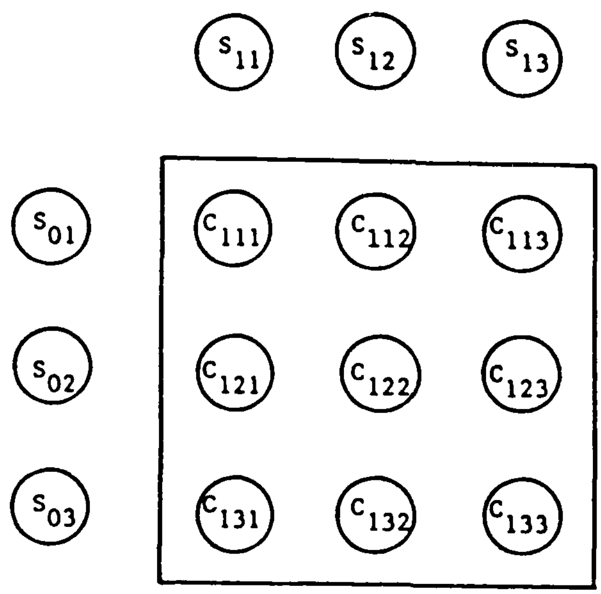

Figure 4-2. Transition Cost Matrix

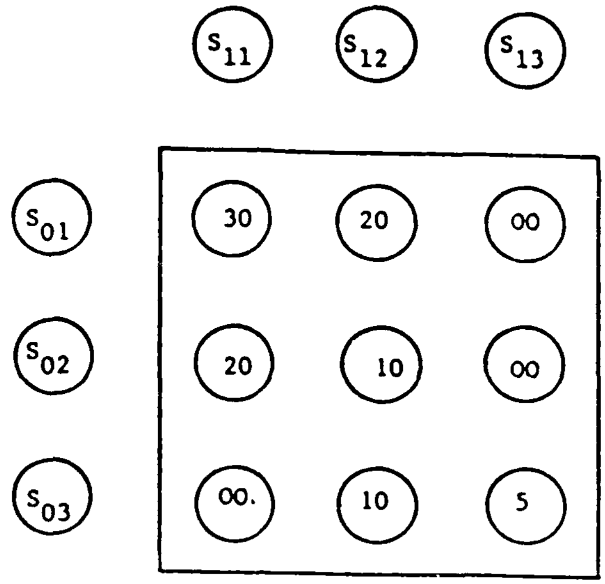

Figure 4-3. Example Transition Cost Matrix

Section III: Problem Solution

7. Identification of Possible Operating Policies. Once the state space and the transition cost matrices have been constructed, a potential operating policy may be found by starting at a state (static head) at the beginning of the first stage (time interval) and drawing a path through one of the states (represented by the circles) associated with each of the remaining stages (see Figure 4-4). For this example problem, the path involves four different steps. Each step in the path corresponds to a decision to change from one static head (state) at the beginning of each time interval to another static head (state) at the end of the time interval. The cost of each decision may be obtained from the transition cost matrix for each stage. The cost associated with each decision path t can be obtained by simply summing the costs associated with each decision in the path. In mathematical notation this can be expressed as:

$$C_t = \sum_{i}^{I} C_{ijk} \tag{4-3}$$

where $j,k \in (t)$

I = number of stages

For the example operating policy, the total cost can be obtained as follows:

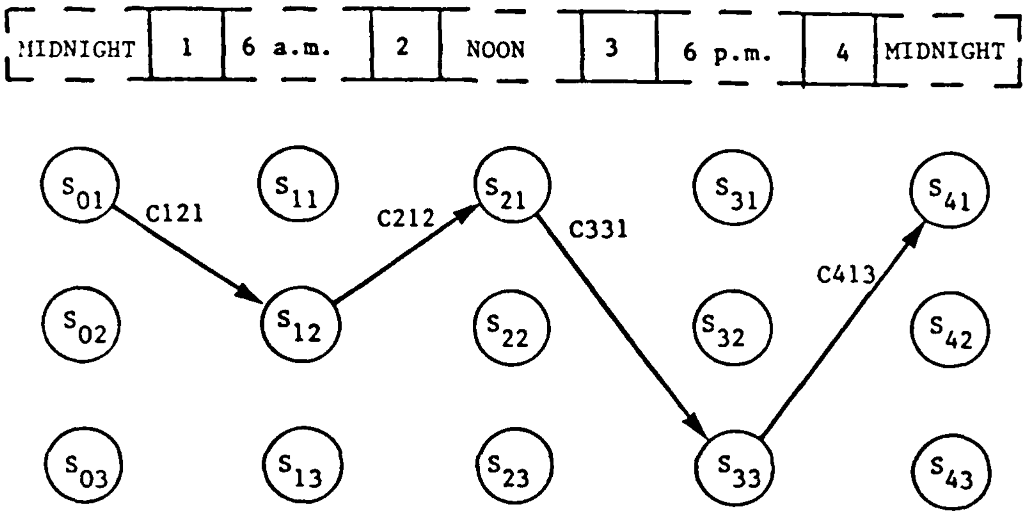

Figure 4-4. Possible Operating Policy

$$C_t = C_{121} + C_{212} + C_{331} + C_{413} \tag{4-4}$$

8. Enumeration Strategy. One possible way to determine the optimal operating policy for the example problem would be to enumerate all the possible paths (operating policies) through the state space. However, even for this small problem there would be $3 \times 3^4 = 243$ such paths involving 972 subproblems (where each subproblem consists of finding the minimum cost required to change from one static head to another). From a practical standpoint many of the paths could be eliminated if the required initial state and the final state are fixed. However, this would still involve 27 paths requiring the solution of 108 subproblems. Therefore, such an approach would quickly become computationally infeasible as the number of states and stages was increased.

9. Dynamic Programming Strategy. Instead of using a complete enumeration strategy to solve the optimal pump operation policy problem, dynamic programming may be used. In order to illustrate the concepts involved in dynamic programming, consider the example problem shown in Figure 4-5. In this case only four stages are considered, where each stage corresponds to a time interval of 6 hr. In addition, the static head at the beginning of the operating period is required to be equal to the static head at the end of the operating period (i.e., 200 ft). During the intermediate time intervals (stages), two additional static heads are possible (i.e., 190 and 210 ft). Instead of constructing transition cost matrices for each stage, the cost associated with each decision may be shown in parentheses directly on the state-space diagram. As before, a cost of infinity indicates a decision that is hydraulically infeasible.

a. Initial Stage. To begin the solution procedure, start at the end of the first stage (see Figure 4-6). For each final state associated with this stage (there are three), enumerate all the possible decisions between the states at the end of the stage and the states at the beginning of the stage (in this case there is only one at the beginning). For each possible decision, determine the associated cost from the transition cost matrix (in this case from Figure 4-5) and record the cost on the state-space diagram as shown in Figure 4-6. Once the cost associated with each decision has been determined, determine the best (least costly) path that can be taken to each final state S_{ik} (associated with stage i) from each initial state $S_{i-1,j}$. The optimal path associated with each final state S_{ik} can be indicated on the state-space diagram by highlighting the paths. The costs associated with each of these optimal decisions should be indicated in brackets next to the corresponding state, as shown in Figure 4-6. For the initial stage there is only one possible path for each of the ending states since there is only one beginning state. The cost in brackets will thus correspond to the cost of each of the individual decisions.

b. Intermediate Stages. For the intermediate stages, again enumerate all the possible decisions (paths) between the states at the end of each stage and the states at the beginning of each stage. For the second and third stages of the example problem, there are nine possible decisions, three decisions for each state, as shown in Figure 4-5. Once again, determine the cost associated with each decision from Figure 4-5. As before, the cost associated with each

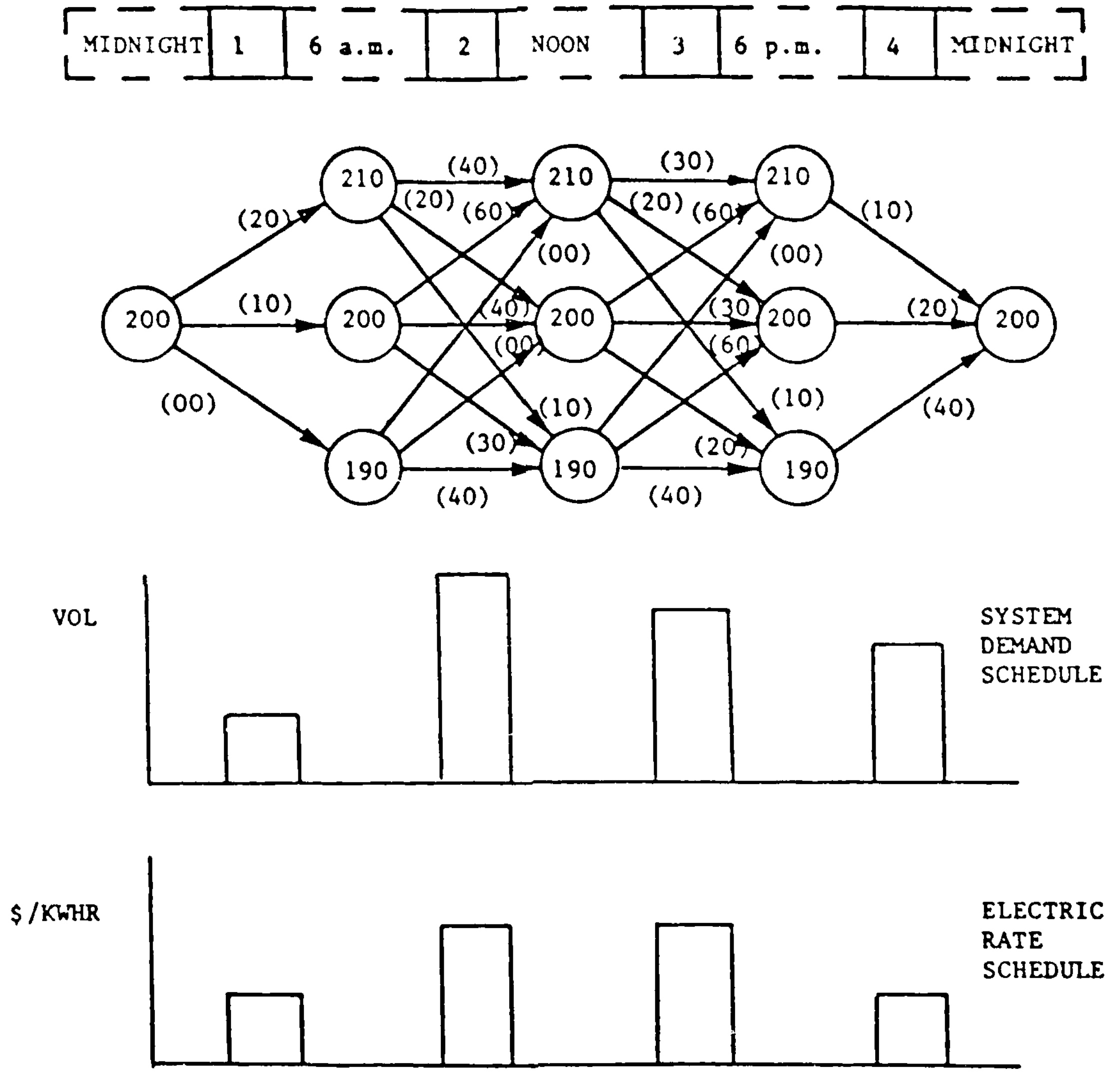

Figure 4-5. Example Problem

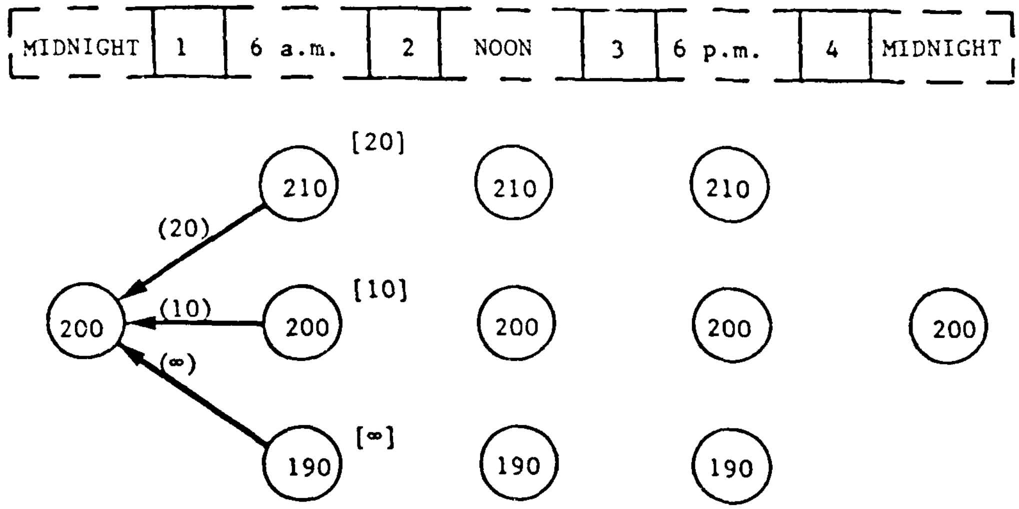

Figure 4-6. Evaluation of Initial Stage

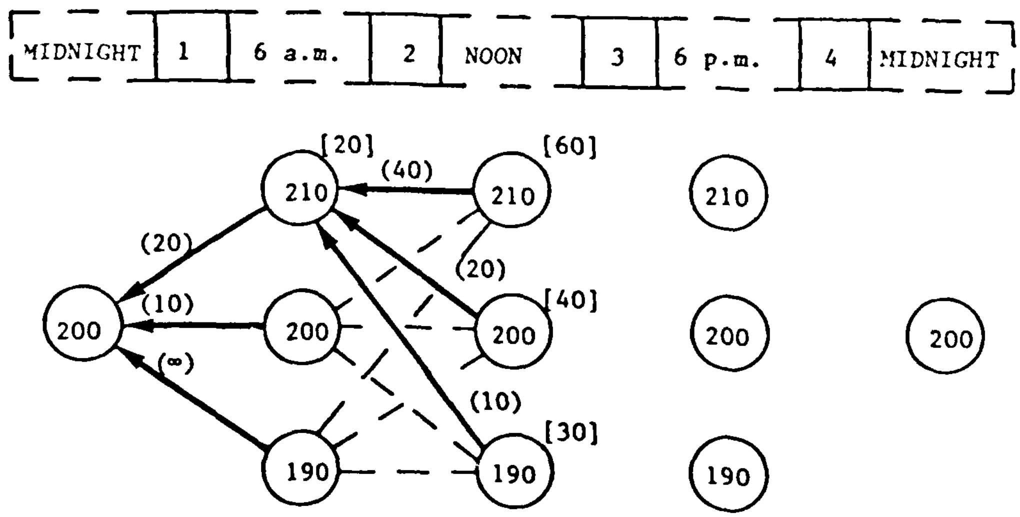

Figure 4-7. Evaluation of Second Stage

decision is shown in parentheses. For each ending state k, determine which previous state j results in the best cumulative decision. This is determined not by looking at the costs associated with the current decision but by looking at the cumulative costs associated with the current decision. The cumulative cost for a particular decision is equal to the sum of the cost of the current decision (in parentheses) and the cumulative cost (in brackets) associated with the previous state of the decision. (This use of the cumulative costs instead of the cost of the current decision is the key to dynamic programming.) Once the cumulative cost for each decision associated with a given state has been determined, find which decision yields the lowest cumulative cost for that state. The decision yielding the lowest cumulative cost is the optimal decision for that state. The optimal decision for each state may be indicated on the state-space diagram by highlighting the path as shown in Figures 4-7 and 4-8. The cumulative cost associated with each optimal decision is now recorded in brackets next to the corresponding state. The optimal solution for stage 2 is shown in Figure 4-7 while the optimal solution for stage 3 is shown in Figure 4-8. By the end of stage 3, many of the alternative decisions have been eliminated.

c. Final Stage. For the final stage, repeat the same steps employed in the previous stages. For the example problem there is only one ending state for the final stage. Once again, determine the cumulative cost associated with each of the three possible decisions. As before, record the minimum of these three in brackets next to the final state (circle). This final cost represents the total cost of the optimal pump operating policy. As before, indicate which decision was used to obtain this cost by highlighting the associated path on the state-space diagram (see Figure 4-9). After the final decision has been determined, the optimal operating policy can be determined by following the emphasized path back through the state space (see Figure 4-10). By following the path back through the state space it is possible to identify the states (static heads) required at each time step that will yield the least-cost operating policy. The optimal solution for the example problem is summarized in Table 4-1.

Table 4-1. Optimal Solution

Time	State	Cost
Midnight	200	--
6 a.m.	210	20
Noon	200	20
6 p.m.	200	30
Midnight	200	20
Total		90

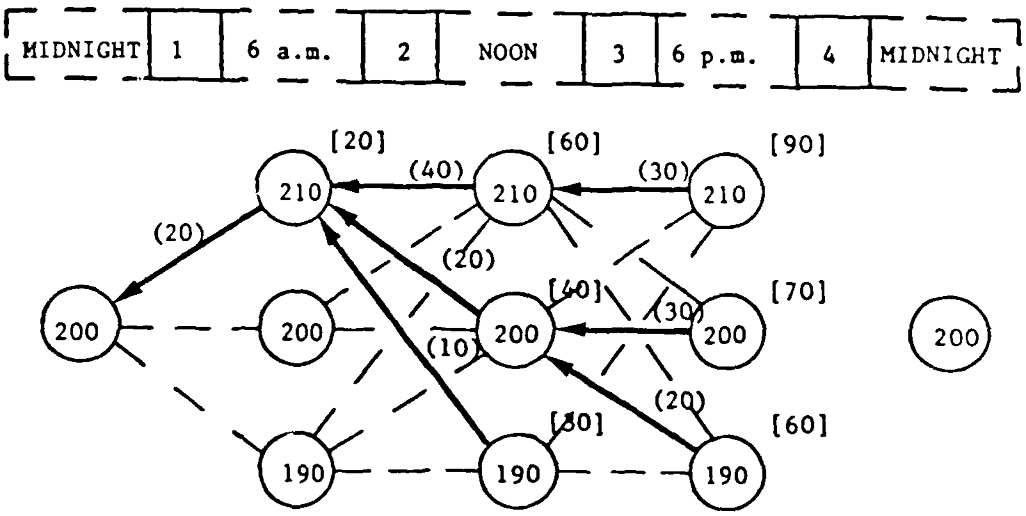

Figure 4-8. Evaluation of Third Stage

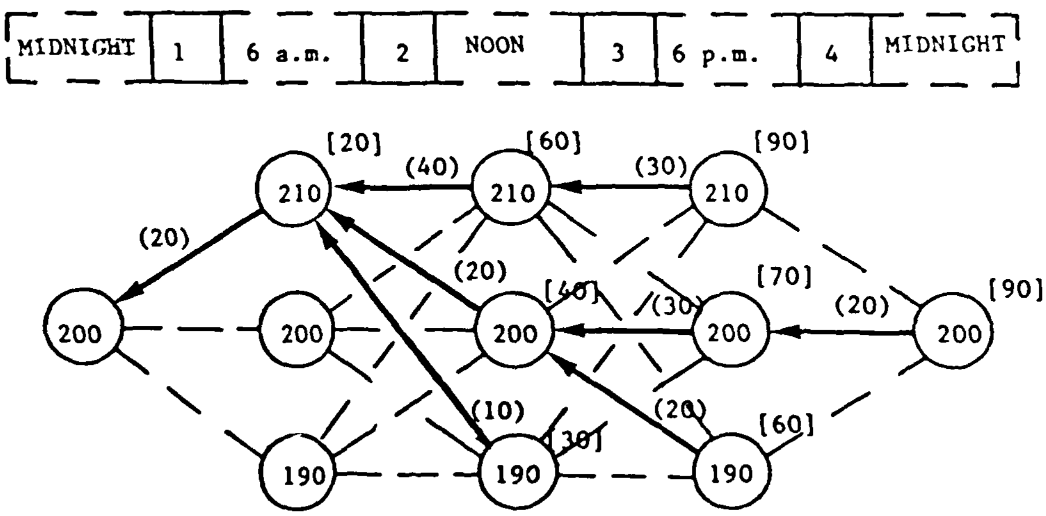

Figure 4-9. Evaluation of Final Stage

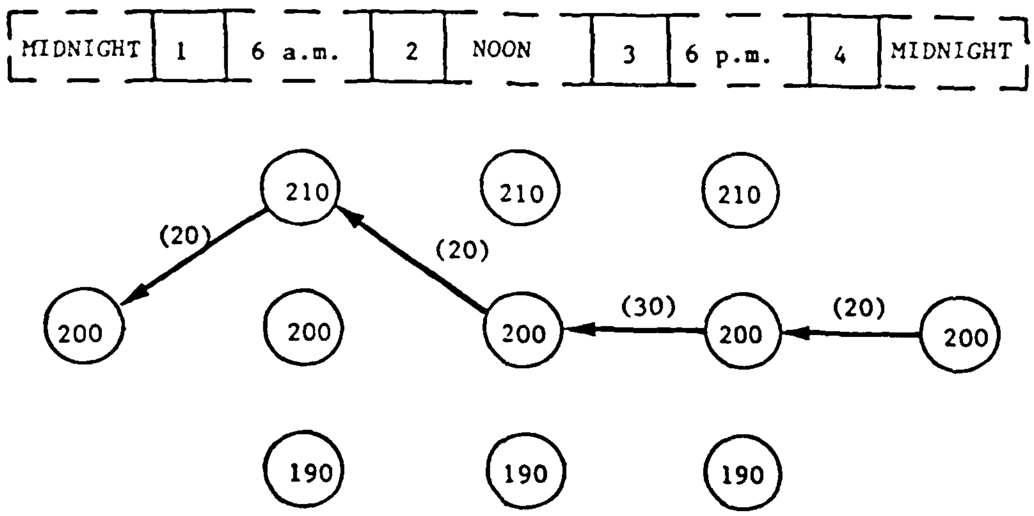

Figure 4-10. Optimal Solution

10. Computer Program. The optimal pump operating policy problem may be solved using dynamic programming. For the example problem discussed in the previous sections, a complete enumeration strategy would require the solution of 108 subproblems. For the same problem, a dynamic programming strategy requires the solution of only 25 subproblems. As the number of states and stages is increased, the computational gap increases exponentially. Although a particular optimal pump operating policy problem could be solved graphically as illustrated in the previous example, such an approach would be extremely tedious and time consuming. (It should be reemphasized that the solution of each subproblem requires the solution of the optimal pump combination problem.) In order to expedite the use of dynamic programming in solving the optimal pump operating policy problem, the entire procedure presented above (including the subproblem solution procedure) has been incorporated into a computer program for easy use.

a. Data Requirements. In order to use the optimal pump operating program (O-POP) the following data are required:

1) Pump head versus discharge curves for each pump.

2) Wire-to-water efficiency curves for each pump.

3) System head curves for each pump or combination of pumps.

4) Average cross-sectional area or volume versus water level table for the elevated storage tanks.

5) Maximum and minimum water surface elevations of the storage tanks.

6) The number of states and stages (time intervals) to be used.

7) The system demand and electric rate for each time interval (stage).

b. Program Operation. Using the pump characteristic curves and the system head curves, the computer program develops a set of pump operation curves for each electric rate using the procedures outlined in Enclosure 3. Using the pump operation curves, the program constructs transition cost matrices for each stage. Once the transition cost matrices have been developed, the program enumerates all the possible transitions between the states associated with each stage and then determines the minimum cumulative cost associated with each state. Once all the stages have been processed and the total minimum cost has been determined, the program moves back through the state space in order to determine the optimal operating policy.

c. Program Results. For a given set of input data the program will determine the most economical average static head (and associated tank water level) for each time interval (stage). In addition, the program will provide a ranking of the various pump combinations that will yield the desired average static head for each time interval. The operator may then use the results to select the combination that is most feasible for the system operation for each time interval.

d. Availability. A computer program to solve the dynamic programming problem has been developed and tested by the US Army Engineer Waterways Experiment Station. The program can run on an IBM-PC or compatible computer. For more information, contact Dr. Thomas M. Walski (WESEE-R), Comm 601-634-3931 or FTS 542-3931.

11. References.

a. Sabet, M. H., and O. J. Helweg, 1985, "Cost Effective Operation of Urban Water Supply System Using Dynamic Programming," Water Resources Bulletin, Vol 21, No. 1, pp 75-81.

b. Sterling, M. J., and B. Coulbeck, 1975a (Dec), "A Dynamic Programming Solution to Optimization of Pumping Costs," Proc. Instn. Civ. Engrs., Part 2, pp 813-818.

c. Sterling, M. J., and B. Coulbeck, 1975b (Dec), "Optimization of Water Pumping Costs by Hierarchical Methods," Proc. Instn. Civ. Engrs., Part 2, pp 789-797.

APPENDIX B: DEVELOPING SYSTEM HEAD CURVES

1. Developing system head curves is a necessary step in pump sizing or evaluating the operation of existing pumps. A simple textbook procedure for developing a system head curve is to: (a) determine the lift required based on the difference in water level between the nearest tanks on the upstream and downstream sides of the pumps, (b) determine friction loss based on head loss between the upstream tank and the pump and between the pump and the downstream tank, and (c) add the heads at each flow rate. Such a system head curve is shown in Figure B1. It is based on the idealized system shown in Figure B2 with the added assumption that $Q_u = 0$. (For convenience, symbols are defined at the conclusion of this appendix.)

2. Virtually all example problems in engineering references (Clark, Viessman, and Hammer 1971; Hicks and Edwards 1971; ASCE 1975; Messina 1976; Reh 1981; Walski 1984) contain system head curves, as shown in Figure B1. These system head curves are based on the assumption that there is virtually no water used or lost between the two tanks. This is a reasonable assumption for many water supply pumps and most sewage pumps. However, in some water distribution systems, very large quantities of water can be consumed between the pump and the nearest tank. In evaluating some in-place pumps in the DC system, the authors observed that actual system head curves could be quite different from those shown in Figure B1.

3. This appendix describes why system head curves can vary significantly from those shown in Figure B1, illustrates some interesting special cases, and describes how engineers can account for these anomalies in practical design situations.

Generalized System Head Curve

4. Consider the water distribution system shown in Figure B2. It differs from those shown as examples in standard engineering texts in that there may be a significant water use (Q_u) at the end of pipe segment 1. The difference in head between the tank upstream of the pump and a point

* See References at the end of this appendix.

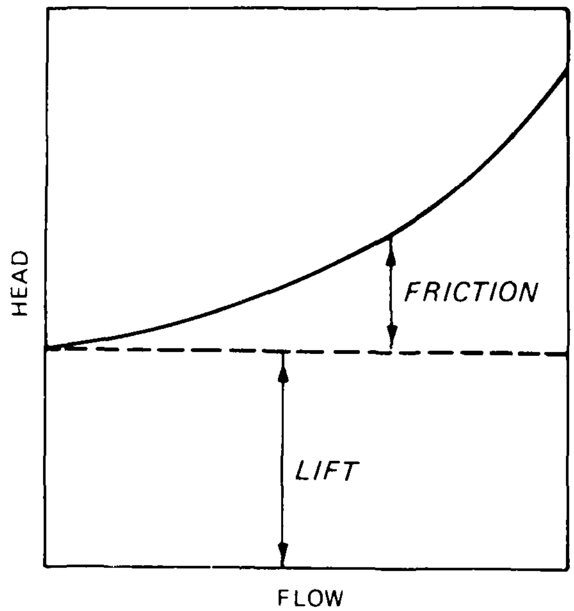

Figure B1. System head curve for idealized system shown in Figure B2

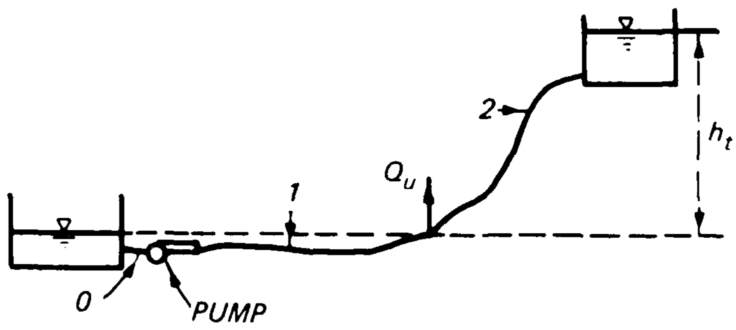

Figure B2. Idealized system

immediately downstream of the pump (i.e., the system head curve) can be given as a function of flow as

$$h = h_t + K_0 Q_0^2 + K_1 Q_1^2 + K_2 |Q_2| Q_2 \qquad \text{(B1)}$$

where

h = head required at pump discharge Q, L

h_t = difference in upstream and downstream tank water levels, L

K_0, K_1, K_2 = head loss coefficients for pipe segments 0, 1, and 2, respectively

Q_0, Q_1, Q_2 = flow in pipe segments 0, 1, and 2, L^3/T

The absolute value term is required in Equation B1 because when Q_u is greater than Q_1, flow will actually occur from the tank toward the pump in pipe segment 2.

5. The best way to illustrate the anomalies in system head curves is to plot several system head curves using reasonable values for head loss coefficients. Such curves are plotted in Figure B3 for a system with $h_t = 50$ ft and $K_0 = 0$ (i.e., short suction line typical of most pumping stations). Note that the horizontal axis is pump discharge, not total water use. Water use downstream of the tank does not affect the curves.

6. The first observation is that only the system head curve for $Q_u = 0$ looks like a typical system head curve. The other curves show unusual shapes for $Q_u > Q_1$. Most notably, the curve for $K_1 = K_2$ and $Q_u = 25$ is a straight line for pump discharges less than 25 ft^3/sec.

7. When the flow in pipe segment 2 is toward the tank (i.e., $Q_2 > 0$), the system head curves show a positive second derivative, as one would expect. However, when Q_2 is negative, the shape of the curves depends on where and how much water is being used between the pumps and the tank.

8. Figure B3 shows that when water use is at the tank ($K_2 = 0$), the system head curves are concave upwards. For water use midway between the tank and the pump ($K_1 = K_2$), the system head curves are linear for small values of pump discharge. For water use at the pump ($K_1 = 0$), the curves are concave downward.

9. This phenomenon can also be explained using the hydraulic grade lines shown in Figure B4. For the traditional system head curve ($Q_u = 0$), the hydraulic grade line slopes downward at a constant slope. As water use between the pump and tank increases ($0 < Q_u < Q_{pump}$), the grade line becomes curved. As water use increases even further ($Q_{pump} < Q_u$), the tank begins to empty, and the hydraulic grade line becomes U-shaped.

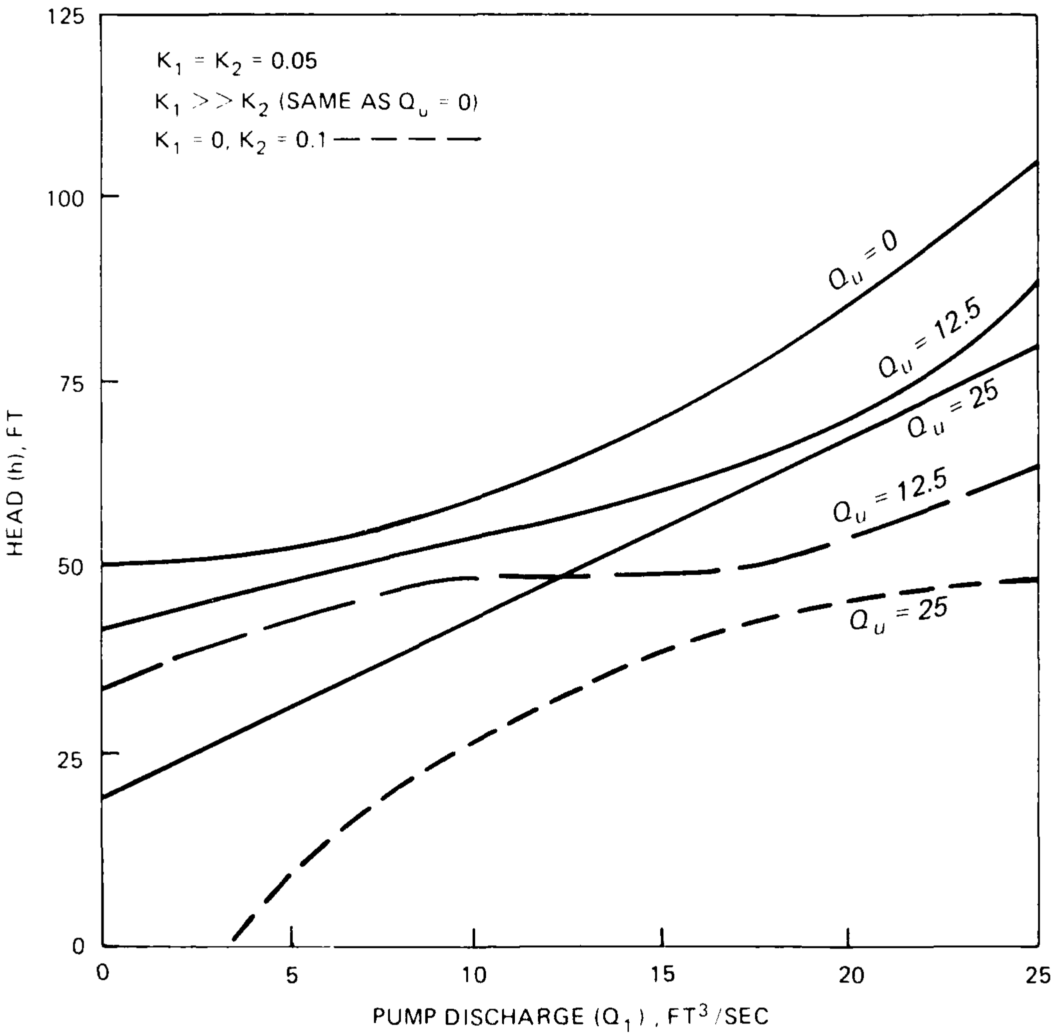

Figure B3. Sample system head curves

Special Cases

10. Some special cases can help illustrate why the odd-shaped system head curves exist. First, consider the case in which both K_0 and K_1 are negligible in comparison with K_2 $(K_0, K_1 << K_2)$. This corresponds to the case in which the water use is very close to the pump. For this case, Equation B1 can be simplified to

$$h = h_t + K_2(Q_2)^2 \qquad \text{(B2a)}$$

when

$$Q_2 \geq 0$$

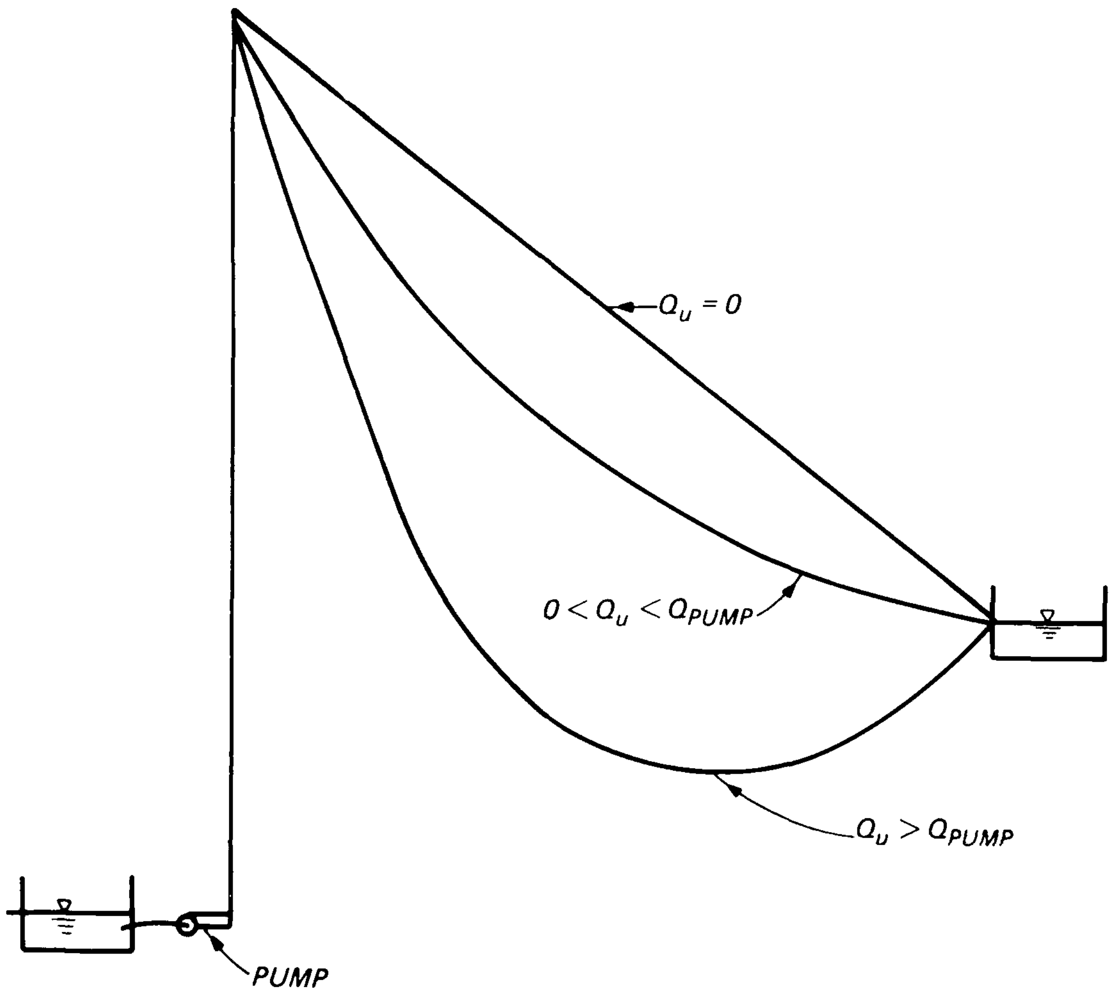

Figure B4. Sample hydraulic grade lines

and

$$h = h_t - K_2(Q_2)^2 \qquad \text{(B2b)}$$

when

$$Q_2 < 0$$

This equation is plotted as Figure B5 and is somewhat startling in that the system head curve is an inverted parabola for small values of pump discharge.

11. The second special case occurs when $K_1 = K_2$ and $K_0 = 0$. For this case, Equation B1 reduces to

$$h = h_t + K_1(2Q_1^2 - 2Q_1Q_u + Q_u^2) \qquad \text{(B3a)}$$

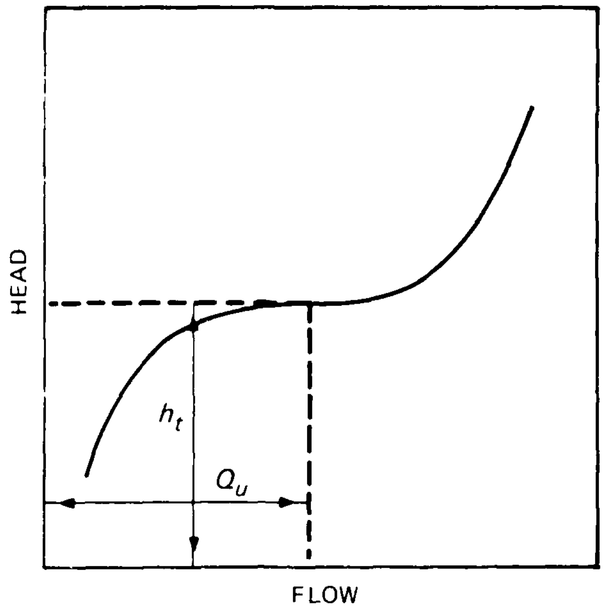

Figure B5. Special case system head curve ($K_0, K_1 << K_2$)

when

$$Q_2 \geq 0$$

and

$$h = h_t + K_1(2Q_1Q_u - Q_u^2) \tag{B3b}$$

when

$$Q_2 < 0$$

Equation B3b is plotted in Figure B6 and illustrates that for some flow rates, the system head curve can be a linear function of pump discharge, Q_1.

Effect on Operating Point

12. Since accounting for water use between the pump and tank moves the system head curve to the right and down, one would expect the operating point of the pump to move in that direction so that the pump would put out greater flow at lower head. Noting that $Q_2 = Q_1 - Q_u$ and that, in most cases, $K_0 << K_1$, the system head curve (Equation B1) can be written using the notation from Figure B2 as

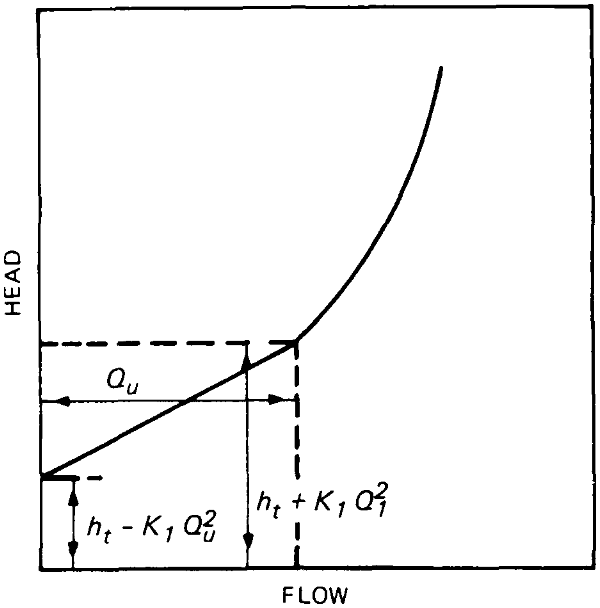

Figure B6. Special case system head curve ($K_0 = 0$, $K_1 = K_2$)

$$h = h_t + K_1 Q_1^2 + K_2 (Q_1 - Q_u) |Q_1 - Q_u| \tag{B4}$$

13. Pump head characteristic curves can be approximated by a parabola as

$$h_p = aQ_1^2 + bQ_1 + c \tag{B5}$$

where

h_p = head produced by pump, L

a, b, c = coefficients in pump curves

In almost all cases, the bQ_1 term is much smaller than the other terms, so to simplify the algebra, it will be eliminated.

14. The operating point of a pump is the combination of flow and head that satisfies Equations B4 and B5 simultaneously (i.e., the intersection of the system head and pump characteristic curves). Setting those two equations equal gives the following equation:

$$(-a + K_1) Q_1^2 + K_2 |Q_1 - Q_u| (Q_1 - Q_u) + h_t - c = 0 \tag{B6}$$

15. It is possible to define a dimensionless parameter $u = Q_u/Q_1$, which gives the relative amount of water consumed between the pump and tank. For $u = 0$, no water is consumed along the line; for $0 < u < 1$, some of the water is consumed; for $u = 1$, exactly all the water pumped is consumed before it reaches the tank; and for $u > 1$, more water is consumed than pumped. Because of the absolute value function in Equation B6, there are two cases to the solution for Q_1:

For $u < 1$,

$$Q_1 = \sqrt{\frac{C - h_t}{K_1 + K_2(u^2 - 2u + 1) - a}} \qquad \text{(B7a)}$$

For $u > 1$,

$$Q_1 = \sqrt{\frac{C - h_t}{K_1 - a - K_2(u^2 - 2u + 1)}} \qquad \text{(B7b)}$$

16. Since $C > h_t$ (or else the pump would not work) and $K_2 > 0$, the minimum value of Q occurs at $u = 0$. As u increases (i.e., more water is used out of the pump discharge line), the discharge from the pump increases. Some of the implications of this are described below:

a. Since the discharge rate is greater and the total volume to be pumped remains constant, pumps will run for shorter periods of time if water is used near the pumps.

b. Since the pump head characteristic curve slopes downward, the pump would produce less head. Th energy cost may or may not decrease, depending on the efficiency of the new operating point. If the pump was designed to operate exactly at the pump's best efficiency point, increasing the flow rate may result in greater energy costs even though the pumps run shorter periods of time against lower heads. This depends on the individual pump's efficiency curve.

c. At the higher flow rates the pump will draw more power. In the case of utilities with a demand charge, the demand charge may be greater than estimated if some significant portion of the water use is near the pump. A larger driver may also be required.

d. Since the net positive suction head (NPSH) required increases nonlinearly with flow and the NPSH available decreases nonlinearly with discharge (due to friction losses in suction piping), a pump that was designed to operate in a safe range

using a standard system head curve may experience cavitation problems if a great deal of water is used near the pump.

Complicated Distribution Systems

17. In the examples above, it was assumed that water consumption between the pump and tank could be lumped at a single point and the pipe network could be represented by two pipe segments with a negligible amount of error. In many cases the piping system is too complicated to use such a simplified approximation to the real system. For example, elevated storage tanks may be located on the opposite side of the demand center from the pumps.

18. In such instances, it is necessary to use a model of the distribution system to assist in generating the system head curve. This can be done by replacing the pump, suction line, and suction storage tank (or clearwell) with a constant input node located at the suction tank at an elevation equal to the water level in that tank. The model would then be run for a number of input flow rates at a fixed water consumption rate. The head at the constant input node would be the ordinate of the system head curve at that flow rate. Alternative system head curves could be generated for different water use rates (e.g., nighttime use, average day, and peak day).

19. The above approach can best be illustrated by an example. Consider the simple system shown in Figure B7 with a tank located at node 32 and a pump at node 10. The water level at the tank is 200 ft above the water level of the suction tank located at node 10. Normal water use is 1,000 gpm with the use distributed fairly evenly among the nodes.

20. First, the water use was fixed at 100 gpm (low-use period), and the input to the system was varied from 0 to 1,500 gpm. The heads corresponding to these inputs are shown as the Use = 100 line in Figure B8. The process was repeated for Use = 1000 (normal use) and Use = 2000 (peak use), also shown on Figure B8. Note that these curves look a great deal like the curves from the special cases described earlier.

Practical Implications

21. While the mathematically interesting special cases above illustrate how unusual system head curves can be shaped, they are unlikely to appear

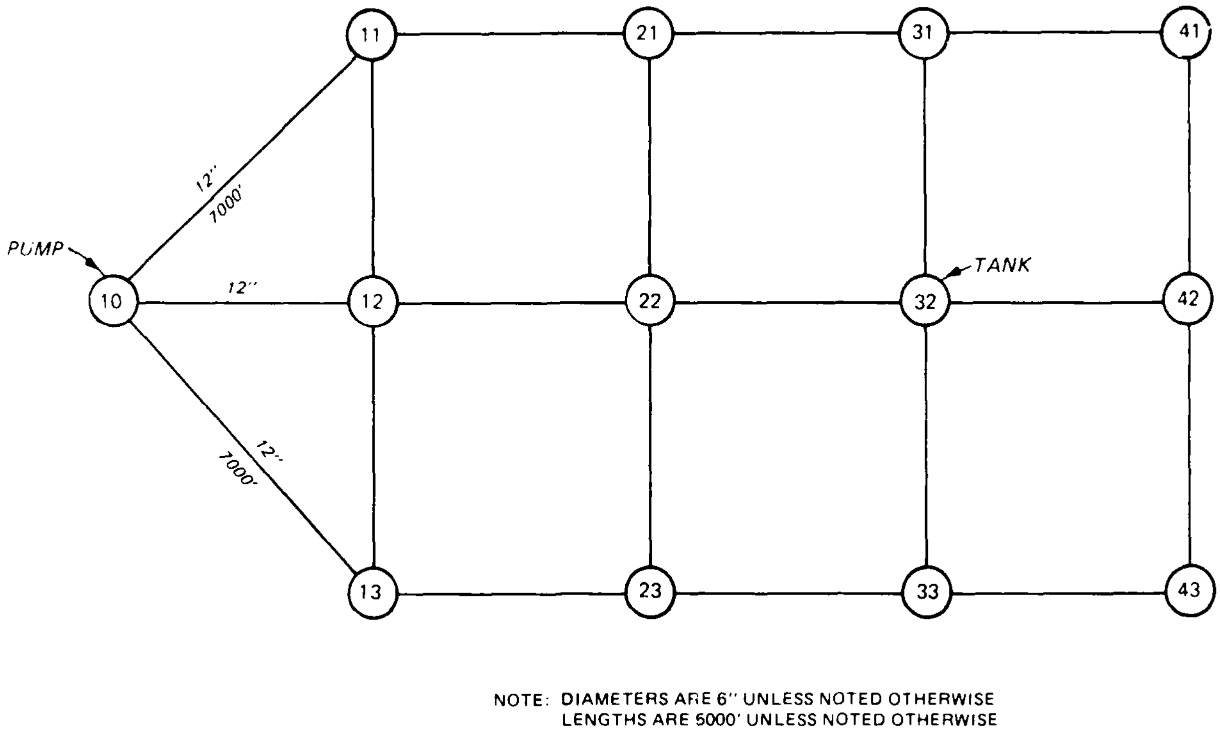

Figure B7. Simple system configuration

exactly as shown in Figures B5 and B6 in actual pump design and operation problems. However, the fact that system head curves are not simply the sum of lift and friction head is significant in water systems where there can be numerous large water users between the pumping station and the nearest tank.

22. One could argue that it is unlikely that pumps will be sized to discharge less water than is being used near the pumping station. However, as more water utilities are facing time-of-day energy pricing in which price of energy is highest in midday when the water use is greatest, the situation of only one or two of a large battery of pumps running at peak-use time is a more common occurrence.

23. The significance of using simple system head curves (e.g. Figure B1) is that engineers will incorrectly determine pump operating points. Pumps that are sized to perform efficiently for traditionally shaped system head curves may behave inefficiently under actual conditions. In addition, the actual pump operating point will be to the left of the expected operating point, so that a pump that was sized to barely meet net positive suction head requirements may actually experience cavitation problems.

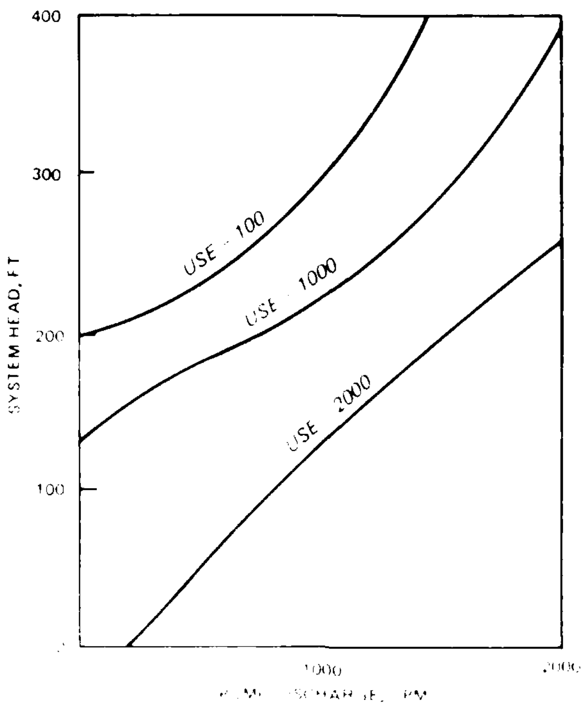

Figure B8. Example system head curves for Figure B7

24. In some cases it is possible to represent a system by a single node, as the authors have done above to generate a system head curve. However, in some cases, this will be a poor approximation to the real system. Instead it will be necessary to generate the system head curve by: (a) setting up a simple skeletal model of the system between the pump and the tank, (b) representing the pump by a constant inflow node, (c) simulating the system for various pump inflows, and (d) determining the required head by subtracting the suction tank water level from the pump head at each inflow. The curves thus generated can be used in pump selection.

Summary

25. The textbook method of generating system head curves by adding lift head and friction head may give misleading results when there is a large water use between the pumps and the downstream tank. This appendix illustrates when such an approach is misleading. Improved system head curves can be generated analytically by lumping water users at a single node or, for more complex systems, by using a water distribution model to determine system heads.

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APPENDIX C: FIELD TEST RESULTS FOR DALECARLIA PUMPING STATION

1. The field test results for the pumps in the Dalecarlia pumping station are summarized in Tables C1-C13. (At the time of the test, Pumps 13 and 14 were out of service and were not tested.) Each table summarizes the physical characteristics of each pump, along with the field test results, which are presented in eight columns.

2. Columns 1, 2, and 5 contain measured data. The first column contains the values of the pump head measured during the test. These values represent the difference in pressure across each pump expressed in feet of water. The pressure on the discharge side of the pump was obtained using a calibrated Bordon tube pressure gage. The pressure on the suction side of the pump was calculated using the elevation of the clearwell, the elevation of the suction line, and the head loss through the suction line. The second column contains the corresponding values of flow rate expressed in units of 1,000 gallons per minute (gpm). These readings were also obtained directly from instrumentation in the control room. The fifth column in each table contains measured values of electrical horsepower. These values were measured directly from instrumentation in the control room.

3. Column 3 contains values of flow rate obtained from the manufacturer's pump characteristic curves. These values were determined using the measured values of pump head. Column 4 contains the percent difference between the measured flow values and the values obtained from the manufacturer's curves.

4. Column 6 contains values of electrical horsepower obtained from the manufacturer's pump characteristic curves. These values were also determined using measured values of pump head. Column 7 contains the percent difference between the measured flow values and the values obtained from the manufacturer's curves.

5. Column 8 contains the wire-to-water efficiencies obtained from the manufacturer's curves using the measured values of pump head.

Table C1

Field Test Results for Dalecarlia Pumping Station, Pump 1

MODEL # 30MC1VERT
SERIAL # 1461376

CLEARWELL 134.5 FT
PUMP ELEV 106 FT
RES ELEV 170 FT

RATED FLOW 34800 GPM
RATED SPEED 514 RPM
RATED HEAD 50 FT

DATE CURVE DEVELOPED: NOV. 9, 1955
CURVE # RY 115346

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
	52	34.38	35.00	-1.79	550	549	.18	83.4
	53	35.07	34.40	1.95	563	550	2.36	83.6
	56	33.68	32.60	3.31	552	550	.36	84.1
	62	28.13	28.60	-1.66	543	537	1.12	84.2
	70	23.96	21.40	11.95	509	495	2.83	77.2
*	72	19.79	20.40	-2.98	503	490	2.65	75.6
*	72	20.49	20.40	.42	509	490	3.88	75.6
*	74	16.84	17.20	-2.09	476	470	1.28	71.0
*	74	16.15	15.00	7.64	476	460	3.48	69.3
*	77	13.19	10.80	22.17	469	440	6.59	62.4

* Pumps 1 and 2 operating in combination.

Table C2

Field Test Results for Dalecarlia Pumping Station, Pump 2

MODEL #	30MC1VERT	CLEARWELL	134.5 FT	RATED FLOW	34800 GPM
SERIAL #	1461375	PUMP ELEV	106 FT	RATED SPEED	514 RPM
		RES ELEV	170 FT	RATED HEAD	50 FT

DATE CURVE DEVELOPED: NOV. 10, 1955
CURVE # RY 115347

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
	54	34.72	33.20	4.59	550	550	.00	82.2
	56	33.33	32.00	4.17	543	550	-1.27	82.4
	61	31.25	28.80	8.51	523	542	-3.51	81.8
	66	29.86	25.40	17.56	523	527	-.76	80.2
	70	24.31	21.60	12.53	485	505	-3.96	75.6
*	72	20.49	20.40	.42	496	497	-.20	73.6
*	74	16.84	17.20	-2.09	472	480	-1.67	68.2
*	77	13.19	10.80	22.17	449	468	-4.06	63.0

* Pumps 1 and 2 operating in combination.

Table [illegible]

Field Test Results for [illegible] Pumping Station, Pump 3

MODEL #	30MC[illegible]	[illegible]EARWELL	[illegible] FT	RATED FLOW	34800 GPM
SERIAL #	[illegible]	PUMP ELEV	[illegible] FT	RATED SPEED	514 RPM
		[illegible] ELEV	[illegible] FT	RATED HEAD	50 FT

DATE CURVE DEVELOPED: SEPT. [illegible]
CURVE # [illegible]

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
52	[illegible]	[illegible]	[illegible]	560	541	3.51	83.6
53	[illegible]	[illegible]	[illegible]	556	541	2.77	83.6
71	[illegible]	[illegible]	[illegible]	[illegible]	493	1.22	73.9
73	[illegible]	[illegible]	[illegible]	[illegible]	468	3.21	65.4

Table [illegible]

Field Test Results for [illegible] Pumping Station, Pump 4

MODEL # 76NA43VR[illegible] SERIAL # 1461379

[illegible] [illegible] FT
PUMP ELEV 166 FT
[illegible] ELEV 246 FT

RATED FLOW 34350 GPM
RATED SPEED 600 RPM
RATED HEAD 145 FT

DATE CURVE DEVELOPED: MARCH [illegible]
CURVE # [illegible]

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
	133	30.56	28.60	6.84	1153	1130	2.04	84.2
	161	21.53	20.80	3.50	1030	1025	.49	82.2
	198	.00	.00	.00	550	535	2.80	UND
*	158	22.57	21.80	3.53	1059	1150	-7.91	85.2
*	180	12.85	13.40	-4.13	831	830	.12	74.8

* Pumps 4 and 6 operating in combination.

Table C5

Field Test Results for Dalecarlia Pumping Station, Pump 5

MODEL #	26NA43VRT	CLEARWELL	134.5 FT	RATED FLOW	34350 GPM
SERIAL #	1461378	PUMP ELEV	106 FT	RATED SPEED	600 RPM
		RES ELEV	246 FT	RATED HEAD	145 FT

DATE CURVE DEVELOPED: JUNE 20, 1959
CURVE # E-175566

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
116	.00	.00	.00	536	460	16.52	UND
132	30.56	31.80	-3.91	1153	1132	1.86	86.6

Table C6

Field Test Results for Dalecarlia Pumping Station, Pump 6

MODEL #	26NA43VRT	CLEARWELL	134.5 FT	RATED FLOW	34350 GPM
SERIAL #	1461377	PUMP ELEV	106 FT	RATED SPEED	600 RPM
		RES ELEV	246 FT	RATED HEAD	145 FT

DATE CURVE DEVELOPED: DEC. 2, 1959
CURVE # E-175770

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
130	30.56	29.30	4.29	1092	1115	-2.06	85.1
161	21.53	21.00	2.51	1005	1003	.20	84.6
186	11.11	9.00	23.46	684	685	-.15	66.0
* 158	22.57	22.00	2.59	1046	1022	2.35	85.2
* 180	12.85	13.40	-4.13	811	810	.12	74.8

* Pumps 4 and 6 operating in combination.

Table C7

Field Test Results for Dalecarlia Pumping Station, Pump 7

MODEL #	18NA33VRT	CLEARWELL	134.5 FT	RATED FLOW	13900 GPM
SERIAL #	1461382	PUMP ELEV	106 FT	RATED SPEED	900 RPM
		RES ELEV	329.7 FT	RATED HEAD	220 FT

DATE CURVE DEVELOPED: MARCH 11, 1959
CURVE # E-175493

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
198	18.40	16.20	13.60	925	1110	-16.67	69.6
260	9.03	.00	UND	382	363	5.23	UND
* 221	14.58	14.90	-2.13	891	990	-10.00	84.2

* Pumps 7 and 9 operating in combination.

Table C8

Field Test Results for Dalecarlia Pumping Station, Pump 8

MODEL #	18NA33VRT	CLEARWELL	134.5 FT	RATED FLOW	13900 GPM
SERIAL #	1461381	PUMP ELEV	106 FT	RATED SPEED	900 RPM
		RES ELEV	331.2 FT	RATED HEAD	220 FT

DATE CURVE DEVELOPED: MARCH 12, 1959
CURVE # E-175494

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
198	18.40	16.70	10.20	912	1015	-10.15	82.2
251	13.89	10.80	28.60	771	840	-8.21	81.6
265	10.42	.00	UND	429	365	17.53	UND
* 214	15.28	15.50	-1.43	885	994	-10.97	84.6
* 258	7.29	7.60	-4.06	402	690	-41.74	72.0
* 263	5.21	.00	UND	349	365	-4.38	UND

* Pumps 8 and 9 operating in combination.

Table C9

Field Test Results for Dalecarlia Pumping Station, Pump 9

MODEL #	18NA33VRT	CLEARWELL	134.5 FT	RATED FLOW	13900 GPM
SERIAL #	1461380	PUMP ELEV	106 FT	RATED SPEED	900 RPM
		RES ELEV	[illegible] FT	RATED HEAD	220 FT

DATE CURVE DEVELOPED: MARCH [illegible]
CURVE # F-175495

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHI (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
	196	[illegible]	[illegible]	[illegible]	[illegible]	1130	-13.98	81.5
	253	[illegible]	[illegible]	[illegible]	[illegible]	820	-5.24	81.0
	263	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-11.96	UND
*	221	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-4.70	85.0
**	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-14.57	84.5
**	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	4.19	75.8
**	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-[illegible].96	UND

* Pumps [illegible]
** Pumps [illegible]

						Percent Difference	(8) Efficiency from Curve (7)
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	85.3
* [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	85.7
** [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-1.86	85.7
*** [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1670	1.14	84.8

* Pumps 10 and 11 operating in combination.
** Pumps 10 and 12 operating in combination.
*** Pumps 10, 11, and 12 operating in combination.

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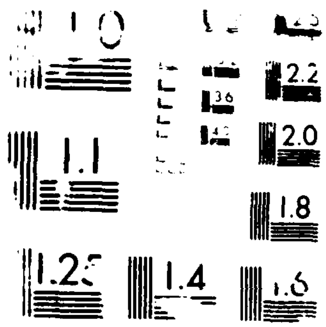
1.0
2.2
2.0
1.1
1.8
1.25
1.4
1.6

Table C11

Field Test Results for Dalecarlia Pumping Station, Pump 11

MODEL #	24NA38VRT	CLEARWELL	134.5 FT	RATED FLOW	18750 GPM
SERIAL #	1461384	PUMP ELEV	106 FT	RATED SPEED	900 RPM
		RES ELEV	423.2 FT	RATED HEAD	300 FT

DATE CURVE DEVELOPED: JUNE 18, 1959
CURVE # E-175568

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
281	23.61	22.90	3.11	1903	1900	.16	84.9
350	11.46	6.60	73.61	1019	1130	-9.82	UND
* 295	12.85	21.90	-41.34	1877	1860	.91	85.9
** 309	15.28	20.10	-23.99	1810	1790	1.12	85.6
*** 325	10.42	17.30	-39.79	1649	1680	-1.85	84.2

* Pumps 10 and 11 operating in combination.
** Pumps 11 and 15 operating in combination.
*** Pumps 10, 11, and 12 operating in combination.

Table C12

Field Test Results for Dalecarlia Pumping Station, Pump 12

MODEL #	24NA38VRT	CLEARWELL	134.5 FT	RATED FLOW	18750 GPM
SERIAL #	1461383	PUMP ELEV	106 FT	RATED SPEED	900 RPM
		RES ELEV	422.5 FT	RATED HEAD	300 FT

DATE CURVE DEVELOPED: JUNE 19, 1959
CURVE # E-175569

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
288	24.31	21.70	12.01	1850	1870	-1.07	84.8
* 302	15.28	20.30	-24.74	1810	1818	-.44	85.3
** 302	15.28	20.30	-24.74	1810	1818	-.44	85.3
*** 325	10.42	17.00	-38.73	1676	1657	1.15	84.7

* Pumps 10 and 12 operating in combination.
** Pumps 12 and 15 operating in combination.
*** Pumps 10, 11, and 12 operating in combination.

Table C13

Field Test Results for Dalecarlia Pumping Station, Pump 15

MODEL #	24NA38VRT	CLEARWELL	134.5 FT	RATED FLOW	18750 GPM
SERIAL #	1461386	PUMP ELEV	106 FT	RATED SPEED	900 RPM
		RES ELEV	422.5 FT	RATED HEAD	300 FT

DATE CURVE DEVELOPED: JUNE 24, 1959
CURVE # E-175572

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Flow from Curve (1000 gpm)	(4) Percent Difference	(5) Measured EHP (HP)	(6) EHP from Curve (HP)	(7) Percent Difference	(8) Efficiency from Curve (%)
	306	24.31	19.30	25.94	1875	1770	5.93	84.5
	353	10.42	.00	UND	885	720	22.92	UND
*	309	15.28	18.90	-19.17	1810	1757	3.02	84.4
**	302	15.28	19.80	-22.84	1810	1790	1.12	84.7

* Pumps 11 and 15 operating in combination.
** Pumps 12 and 15 operating in combination.

APPENDIX D: FIELD TEST RESULTS FOR BRYANT STREET PUMPING STATION

1. The field test results for pumps in the Bryant Street pumping station are summarized in Tables D1-D11. (Because of operational conditions, Pump 1 was not tested individually.) Each table summarizes the physical characteristics of each pump, along with the field test results, which are presented in eight columns.

2. Column 1 contains the values of the pump head measured during the test. These values represent the difference in pressure across each pump expressed in feet of water. The pressure on the discharge side of the pump was obtained using a calibrated Bordon tube pressure gage. The pressure on the suction side of the pump was calculated using the elevation of the clearwell, the elevation of the suction line, and the head loss through the suction line. Column 2 contains the corresponding values of flow rate expressed in units of 1,000 gallons per minute (gpm). Column 3 in each table contains measured values of current, expressed in amperes. Column 4 contains measured values of power expressed in kilovolts-ampere reactive (kvars). The current and power readings were obtained directly from instrumentation in the control room. For an associated voltage drop, the corresponding electrical power in kilowatts may be obtained using the following equation:

$$\text{Power (kW)} = \sqrt{(\text{kVA})^2 - (\text{kvar})^2}$$

where kVA = current (amps) * voltage (volts) . The resulting values of electrical power expressed in units of horsepower are given in column 5.

3. Column 6 contains values of wire-to-water efficiency associated with the head, flow rate, and power readings in columns 1, 2, and 5. Column 7 contains values of wire-to-water efficiency obtained from the manufacturer's pump curves using the measured pump head values in column 1. The percent difference between the calculated wire-to-water efficiencies and the manufacturer's wire-to-water efficiency is shown in column 8.

Table D1

Field Test Results for Bryant Street Pumping Station, Pump 1

MODEL #	26NAS43VT	CLEARWELL	156.5 FT	RATED FLOW	24300 GPM
MODEL #	26NAS43VT	PUMP ELEV	118 FT	RATED SPEED	514 RPM
		RES ELEV	250 FT	RATED HEAD	110 FT
				RATED POWER	800 HP

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
*	96	25.00	150	10	597.47	75.86	N/A	N/A

* Pumps 1 and 3 operating in combination.

Table D2

Field Test Results for Bryant Street Pumping Station, Pump 2

MODEL #	26NAS43VT	CLEARWELL	156.5 FT	RATED FLOW	24300 GPM
SERIAL #	1346786	PUMP ELEV	118 FT	RATED SPEED	514 RPM
		RES ELEV	250 FT	RATED HEAD	110 FT
				RATED POWER	800 HP

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
93	25.76	175	20	696.86	64.93	N/A	N/A
94	24.17						
102	22.84						
132	18.14						
135	14.68	130	160	492.55	75.71	N/A	N/A
136	11.54						
142	8.25						
* 98	24.22	172	80	680.51	65.77	N/A	N/A
* 104	20.90						
* 116	18.44	165	100	649.66	62.23	N/A	N/A
* 129	15.80						
* 132	11.79	130	200	477.71	61.61	N/A	N/A

* Pumps 2 and 3 operating in combination.

Table D3

Field Test Results for Bryant Street Pumping Station, Pump 3

MODEL #	26NAS43VT	CLEARWELL	156.5 FT	RATED FLOW	24300 GPM
SERIAL #	1346784	PUMP ELEV	118 FT	RATED SPEED	514 RPM
		RES ELEV	250 FT	RATED HEAD	110 FT
				RATED POWER	800 HP

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
	93	27.78	170	80	672.49	72.55	N/A	N/A
	141	.00	240	85	952.31	.00	N/A	N/A
*	96	26.39	150	140	580.93	82.35	N/A	N/A
**	98	41.04	155	30	616.75	76.98	N/A	N/A
**	104	42.71						
**	132	51.39	160	30	636.69	95.31	N/A	N/A

* Pumps 1 and 3 operating in combination.
** Pumps 2 and 3 operating in combination.

Table D4

Field Test Results for Bryant Street Pumping Station, Pump 4

MODEL #	30MC1VRT	CLEARWELL	156.5 FT	RATED FLOW	24300 GPM
SERIAL #	1346781	PUMP ELEV	118 FT	RATED SPEED	514 RPM
		RES ELEV	NONE FT	RATED HEAD	45 FT
				RATED POWER	325 HP

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
	11	37.50	60	25	237.71	33.20	N/A	N/A
	77	.00	49	50	188.69	.00	N/A	N/A
*	24	31.25	66	10	262.74	53.52	N/A	N/A

* Pumps 4 and 5 operating in combination.

Table D5

Field Test Results for Bryant Street Pumping Station, Pump 5

MODEL #	30MC1VRT	CLEARWELL	156.5 FT	RATED FLOW	24300 GPM
SERIAL #	1346782	PUMP ELEV	118 FT	RATED SPEED	514 RPM
		RES ELEV	NONE FT	RATED HEAD	45 FT
				RATED POWER	325 HP

	(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
	11	37.50	35	69	121.16	65.14	N/A	N/A
	45	27.78	74	100	277.32	84.33	N/A	N/A
	79	.00	64	140	213.08	.00	N/A	N/A
*	24	31.94	72	70	278.15	51.68	N/A	N/A

* Pumps 4 and 5 operating in combination.

Table D6

Field Test Results for Bryant Street Pumping Station, Pump 6

MODEL #	30MC1VRT	CLEARWELL	156.5 FT	RATED FLOW	24300 GPM
SERIAL #	1346783	PUMP ELEV	118 FT	RATED SPEED	514 RPM
		RES ELEV	NONE FT	RATED HEAD	45 FT
				RATED POWER	325 HP

DATE CURVE DEVELOPED: JULY 19, 1951
CURVE # E-121814

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
12	38.89	60	60	231.37	39.04	37	5.50
17	37.57	60	70	228.54	52.49	41.5	26.49
38	32.28	66	50	258.13	88.94	78	14.02
54	30.13						
63	23.19						
84	.00	56	80	208.25	.00	0	UND

Table D7

Field Test Results for Bryant Street Pumping Station, Pump 7

MODEL #	24MA38VRT	CLEARWELL	156.5 FT	RATED FLOW	17350 GPM
SERIAL #	1346787	PUMP ELEV	118 FT	RATED SPEED	720 RPM
		RES ELEV	332.5 FT	RATED HEAD	210 FT
				RATED POWER	1100 HP

DATE CURVE DEVELOPED: DEC. 15, 1950
CURVE # E-121303

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
183	16.20	210	-130	826.42	67.73	85	-20.32
186	16.07	210	-110	829.32	67.78	82.5	-17.84
188	17.79	200	-110	789.11	79.85	83	-3.79
253	3.92	80	110	299.11	62.34	46.5	34.07

Table D8

Field Test Results for Bryant Street Pumping Station, Pump 8

MODEL #	24MA38VRT	CLEARWELL	156.5 FT	RATED FLOW	17350 GPM
SERIAL #	1346788	PUMP ELEV	118 FT	RATED SPEED	720 RPM
		RES ELEV	332.5 FT	RATED HEAD	210 FT
				RATED POWER	1100 HP

DATE CURVE DEVELOPED: JAN. 4, 1951
CURVE # E-121347

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measure Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
176	15.28	230	150	903.89	56.18	79.3	-29.15
250	.00	160	370	519.01	.00	N/A	N/A

Table D9

Field Test Results for Bryant Street Pumping Station, Pump 9

MODEL #	18NA37VRT		CLEARWELL	156.5 FT		RATED FLOW	10040 GPM
SERIAL #	1346789		PUMP ELEV	118 FT		RATED SPEED	900 RPM
			RES ELEV	420.7 FT		RATED HEAD	310 FT
						RATED POWER	1000 HP

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
269	11.81	200	120	787.65	75.93	N/A	N/A
343	3.47	185	200	709.33	31.62	N/A	N/A
352	.00	110	340	276.45	.00	N/A	N/A

Table D10

Field Test Results for Bryant Street Pumping Station, Pump 10

MODEL #	18NA37VRT	CLEARWELL	156.5 FT	RATED FLOW	10040 GPM		
SERIAL #	1346790	PUMP ELEV	118 FT	RATED SPEED	900 RPM		
		RES ELEV	420.7 FT	RATED HEAD	310 FT		
				RATED POWER	1000 HP		

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
273	10.82	210	160	821.14	67.90	N/A	N/A
280	10.72	205	190	794.25	71.28	N/A	N/A
317	10.28	190	210	727.19	84.58	N/A	N/A
343	6.14	140	300	470.16	84.34	N/A	N/A
350	3.23	130	300	422.14	50.41	N/A	N/A
352	2.43	105	320	269.38	59.85	N/A	N/A

Table D11

Field Test Results for Bryant Street Pumping Station, Pump 11

MODEL # SIZE 20x18 TYPE SG		CLEARWELL	156.5 FT	RATED FLOW	13900 GPM
SERIAL # 23564 23565		PUMP ELEV	118 FT	RATED SPEED	720 RPM
		RES ELEV	420.7 FT	RATED HEAD	155 FT
				RATED POWER	325 HP

(1) Measured Head (ft)	(2) Measured Flow (1000 gpm)	(3) Measured Current (AMPS)	(4) Measured Power (KVARS)	(5) Measured Power (HP)	(6) Computed Efficiency (%)	(7) Efficiency from Curve (%)	(8) Percent Difference
283	11.81	240	210	932.74	67.42	N/A	N/A
387	.00	155	400	470.40	.00	N/A	N/A

APPENDIX E: PCP - PUMP COMBINATION PROGRAM

1. The pump combination program enumerates and ranks the various feasible pump combinations required to meet a specified system demand for a given average tank level (over time) and may be used to develop cost operation curves for use with the tank operation program (TOP). To rank the various pump combinations, five main types of input data are required: the physical description of the controlling tank, the initial and final tank levels, the time interval, the average system demand, and a set of pump operation curves for each pump combination.

2. Two sets of three pump operation curves are required as input for each pump combination. The first set of curves (TLF curves) is used to approximate the hydraulics of the system for each different pump combination. Each curve describes the variation in pump flow for an associated tank level and system demand. The second set of pump operation curves (TLC curves) is used to approximate the pump operation costs associated with each different pump combination. Each curve describes the variation in cost for an associated tank level and system demand. In the computer program, each curve is obtained by fitting a quadratic curve through three different data points supplied by the user.

3. The pump combination program is written in Fortran and is run in a batch mode. Input to the program is read from a user-supplied data file, and output is directed to a user-supplied output file. Data input instructions for PCP are given in Table E1. The first card identifies the duration of the specified operating period, the electric rate and average system demand for the period, and the number of pump combinations to be considered. The second card is used to specify the tank area, the initial and final tank elevations, and the three system demands that correspond to the three pump operation curves.

4. Cards N1, Q1, C1, Q2, C2, Q3, and C3 are repeated for each pump combination. Card N1 identifies each pump combination with a set of numbers. For each combination, the associated pumps are indicated by inputing a nonnegative number in the corresponding data field.

5. The next three sets of cards are used to describe the pump operation curves (TLF and TLC curves) for each pump combination. Cards Q1 and C1 are

Table E1

Data Input Instructions for PCP

Card Group	Format	Column	Description	Variable Name
C1	2X	1-2	Card Group Identifier	
	F8.0	3-10	Time Interval (hr)	DTIME
	F10.0	11-20	Electric Rate (¢/kWhr)	PKWHR
	F10.0	21-30	System Demand (gpm)	QDEM
	I10.0	31-40	Number of Pump Combinations	NCOMB
	I10.0	41-50	Debug Flag	IBUG
C2	2X	1-2	Card Group Identifier	
	F10.0	3-10	Total Area of Tanks (sq ft)	ATANK
	F10.0	11-20	Initial Tank Elevation (ft)	ETANK
	F10.0	21-30	Final Tank Elevation (ft)	FTANK
	F10.0	31-40	Maximum System Demand (gpm)	QMAXD
	F10.0	41-50	Medium System Demand (gpm)	QMIDD
	F10.0	51-60	Minimum System Demand (gpm)	QMIND
			Repeat Cards N1, Q1, C1, Q2, C2, Q3, C3 for each pump combination I = 1, NCOMB	
N1	2X	1-2	Card Group Identifier	
	I3.0	3-5	Pump 1 ID #	NNPC(I,1)
	I5.0	6-10	Pump 2 ID #	NNPC(I,2)
	I5.0	11-15	Pump 3 ID #	NNPC(I,3)
	I5.0	16-20	Pump 4 ID #	NNPC(I,4)
	I5.0	21-25	Pump 5 ID #	NNPC(I,5)

(Continued)

(Sheet 1 of 4)

Table E1 (Continued)

Card Group	Format	Column	Description	Variable Name
	I10.0	26-30	Pump 6 ID #	NNPC(I,6)
	I10.0	31-35	Pump 7 ID #	NNPC(I,7)
	I10.0	36-40	Pump 8 ID #	NNPC(I,8)
			Static Head vs. Flow Rate Curve for QMAXD	
Q1	2X	1-2	Card Group Identifier	
	F8.0	3-10	First Static Head (ft)	X1
	F10.0	11-20	First Flow Rate (gpm)	Y1
	F10.0	21-30	Second Static Head (ft)	X2
	F10.0	31-40	Second Flow Rate (gpm)	Y2
	F10.0	41-50	Third Static Head (ft)	X3
	F10.0	51-60	Third Flow Rate (gpm)	Y3
			Static Head vs. Unit Cost Curve for QMAXD	
C1	2X	1-2	Card Group Identifier	
	F8.0	3-10	First Static Head (ft)	X1
	F10.0	11-20	First Unit Cost (¢/kgal/hr)	Y1
	F10.0	21-30	Second Static Head (ft)	X2
	F10.0	31-40	Second Unit Cost (¢/kgal/hr)	Y2
	F10.0	41-50	Third Static Head (ft)	X3
	F10.0	51-60	Third Unit Cost (¢/kgal/hr)	Y3

(Continued)

(Sheet 2 of 4)

Table E1 (Continued)

Card Group	Format	Column	Description	Variable Name
			Static Head vs. Flow Rate Curve for QMIDD	
Q2	2X	1-2	Card Group Identifier	
	F8.0	3-10	First Static Head (ft)	X1
	F10.0	11-20	First Flow Rate (gpm)	Y1
	F10.0	21-30	Second Static Head (ft)	X2
	F10.0	31-40	Second Flow Rate (gpm)	Y2
	F10.0	41-50	Third Static Head (ft)	X3
	F10.0	51-60	Third Flow Rate (gpm)	Y3
			Static Head vs. Unit Cost Curve for QMIDD	
C2	2X	1-2	Card Group Identifier	
	F8.0	3-10	First Static Head (ft)	X1
	F10.0	11-20	First Unit Cost (¢/kgal/hr)	Y1
	F10.0	21-30	Second Static Head (ft)	X2
	F10.0	31-40	Second Unit Cost (¢/kgal/hr)	Y2
	F10.0	41-50	Third Static Head (ft)	X3
	F10.0	51-60	Third Unit Cost (¢/kgal/hr)	Y3
			Static Head vs. Flow Rate Curve for QMIND	
Q3	2X	1-2	Card Group Identifier	
	F8.0	3-10	First Static Head (ft)	X1
	F10.0	11-20	First Flow Rate (gpm)	Y1

(Continued)

(Sheet 3 of 4)

Table E1 (Concluded)

Card Group	Format	Column	Description	Variable Name
	F10.0	21-30	Second Static Head (ft)	X2
	F10.0	31-40	Second Flow Rate (gpm)	Y2
	F10.0	41-50	Third Static Head (ft)	X3
	F10.0	51-60	Third Flow Rate (gpm)	Y3
			Static Head vs. Unit Cost Curve for QMIND	
C3	2X	1-2	Card Group Identifier	
	F8.0	3-10	First Static Head (ft)	X1
	F10.0	11-20	First Unit Cost (¢/kgal/hr)	Y1
	F10.0	21-30	Second Static Head (ft)	X2
	F10.0	31-40	Second Unit Cost (¢/kgal/hr)	Y2
	F10.0	41-50	Third Static Head (ft)	X3
	F10.0	51-60	Third Unit Cost (¢/kgal/hr)	Y3

(Sheet 4 of 4)

used to describe the tank level versus discharge and tank level versus unit cost curves for the operating condition corresponding to a maximum system demand of QMAXD. These cards are used to describe the top curve of the three TLF and TLC curves associated with each pump combination. Cards Q2 and C2 are used to describe the tank level versus discharge and tank level versus unit cost curves for the operating condition corresponding to an average system demand of QMIDD. These cards are used to describe the middle curve of the three TLF and TLC curves associated with each pump combination. Finally, cards Q3 and C3 are used to describe the tank level versus discharge and tank level versus unit cost curves for the operating condition corresponding to an average system demand of QMIND. These cards are used to describe the lower curve of the three TLF and TLC curves associated with each pump combination (see Figures E1 and E2).

6. A complete listing of the program is provided as Figure E3. A typical input data file is shown in Figure E4. A typical output data file is shown in Figure E5.

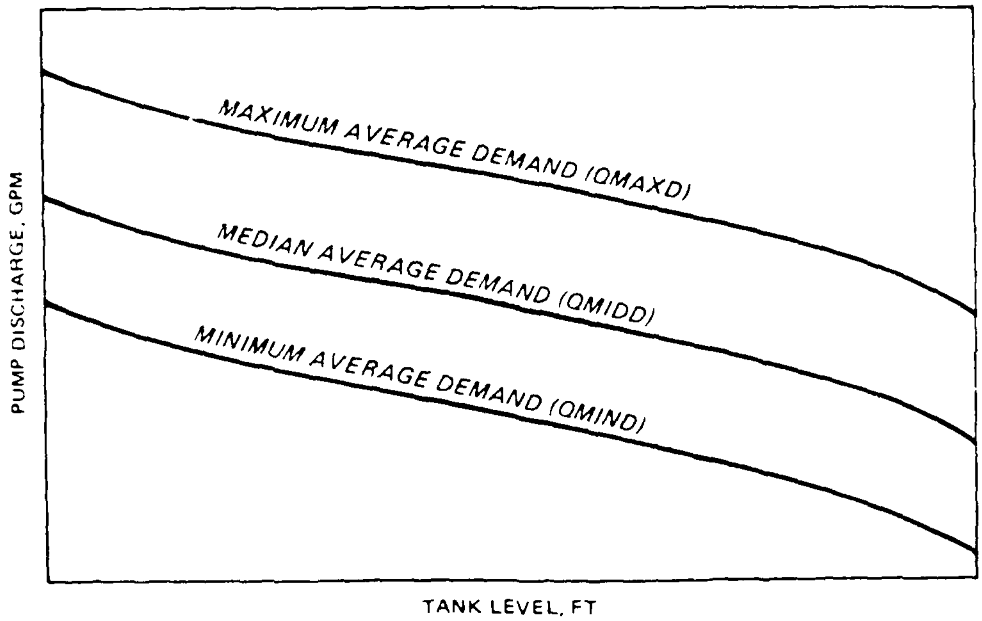

Figure E1. Tank level versus pump discharge (TLF curves)

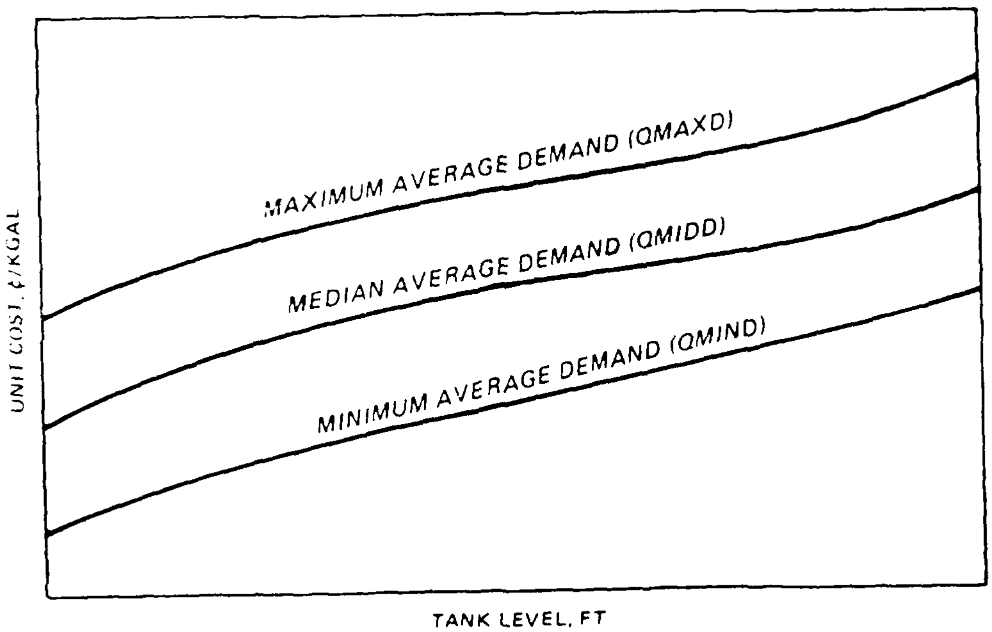

Figure E2. Tank level versus unit cost (TLC curves)

```
C                         **********************************
C                         * PCP - PUMP COMBINATION PROGRAM *
C                         *                                *
C                         * AUTHOR - LINDELL E. ORMSBEE    *
C                         *                                *
C                         * LATEST REVISION 12/1/86        *
C                         **********************************
C
C     THIS PROGRAM WILL DETERMINE THE MOST EFFICIENT COMBINATION
C     OF PUMPS REQUIRED TO SATISFY A GIVEN SYSTEM DEMAND AND AVG
C     STATIC HEAD.
C
C     FOR EACH  COMBINATION OF PUMPS A STATIC HEAD VS DISCHARGE
C     AND A STATIC HEAD VS UNIT COST CURVE ARE REQUIRED.  THESE
C     CURVES  MAY BE  GENERATED USING  THE PROCEDURES  GIVEN IN
C     APPENDIX C.
C
      COMMON/BLK1/ X0,Y0,X1,Y1,X2,Y2,C1,C2,C3
C
      DIMENSION NNPC(100,6),QP(100),CP(100),CCST(100)
      DIMENSION QX(21,3),CX(21,3),QM(21,3),CM(21,3),QN(21,3),CN(21,3)
      DIMENSION QB(100),CB(100),QA(100),CA(100),IAP(100),IBP(100)
      DIMENSION IAC(100),IBC(100),QCA(100),QCB(100),FCA(100),FCB(100)
      DIMENSION CCA(100),CCB(100),CRA(100),CRB(100)
C
      CHARACTER*14 FILOT,FILIN
C
      REAL X0,Y0,X1,Y1,X2,Y2
C
  201 FORMAT(I10)
  202 FORMAT(F10.0)
C
      WRITE(*,1)
    1 FORMAT('/ INPUT NAME OF INPUT FILE'/)
      READ(*,3)FILIN
      WRITE(*,2)
    2 FORMAT('/ INPUT NAME OF OUTPUT FILE'/)
      READ(*,3)FILOT
    3 FORMAT(A14)
C
      OPEN(5,FILE=FILIN,STATUS='OLD')
      OPEN(6,FILE=FILOT,STATUS='NEW')
C
C     ****************
C     * READ IN DATA *
C     ****************
C
C     DTIME = TIME INTERVAL (HRS)
C     PKWHR = ELECTRIC RATE (C/KWHR)
C     QDEM  = SYSTEM DEMAND
C     NCOMB = NUMBER OF PUMP COMBINATIONS
C     DEBUG = DEBUG FLAG (0=NORMAL OUTPUT, 1=EXTENDED OUTPUT)
C
      READ(5,4)DTIME,PKWHR,QDEM,NCOMB,IBUG
```

Figure E3. Program listing for PCP (Sheet 1 of 7)

```
    4   FORMAT(2X,F8.0,2F10.0,2I10)
C
C       ATANK = AREA OF CONTROLLING TANK
C       ETANK = INITIAL TANK ELEVATION
C       FTANK = FINAL TANK ELEVATION
C       QMAXD = MAXIMUM SYSTEM DEMAND
C       QMIDD = MEDIAN SYSTEM DEMAND
C       QMIND = MINIMUM SYSTEM DEMAND
C
        READ(5,5)ATANK,ETANK,FTANK,QMAXD,QMIDD,QMIND
    5   FORMAT(2X,F8.0,5F10.0)
C
        HAVG=(ETANK+FTANK)/2.0
        QSTR=(FTANK-ETANK)*ATANK*448.8/(DTIME*3600.0)
        QREQ=QDEM-QSTR
        IF(QREQ.LE.0.0)GOTO 200
C
        C1=0.0
        C2=0.0
        C3=0.0
C
        JJ=NCOMB+1
C
        DO 7 I=1,6
        NNPC(JJ,I)=0
    7   CONTINUE
C
C       READ IN STATIC HEAD VS DISCHARGE CURVES X = H, Y = Q
C               STATIC HEAD VS UNIT COST CURVES X = H, Y = C
C               FOR FLOW VALUES OF QMAXD, QMIDD, QMIND
C
C
    8   FORMAT(2X,I3,7I5)
    9   FORMAT(2X,F8.0,5F10.0)
C
        DO 12 I=1,NCOMB
        READ(5,8)(NNPC(I,J),J=1,8)
        READ(5,9)X0,Y0,X1,Y1,X2,Y2
        CALL SCURVE(IBUG)
        QX(I,1)=C1
        QX(I,2)=C2
        QX(I,3)=C3
        READ(5,9)X0,Y0,X1,Y1,X2,Y2
        CALL SCURVE(IBUG)
        CX(I,1)=C1
        CX(I,2)=C2
        CX(I,3)=C3
        READ(5,9)X0,Y0,X1,Y1,X2,Y2
        CALL SCURVE(IBUG)
        QM(I,1)=C1
        QM(I,2)=C2
        QM(I,3)=C3
        READ(5,9)X0,Y0,X1,Y1,X2,Y2
        CALL SCURVE(IBUG)
```

Figure E3. (Sheet 2 of 7)

```
      CM(I,1)=C1
      CM(I,2)=C2
      CM(I,3)=C3
      READ(5,9)X0,Y0,X1,Y1,X2,Y2
      CALL SCURVE(IBUG)
      QN(I,1)=C1
      QN(I,2)=C2
      QN(I,3)=C3
      READ(5,9)X0,Y0,X1,Y1,X2,Y2
      CALL SCURVE(IBUG)
      CN(I,1)=C1
      CN(I,2)=C2
      CN(I,3)=C3
 12   CONTINUE
C
C     ******************************
C     * DETERMINE FLOW AND COST FOR *
C     * EACH PUMP COMBINATION       *
C     ******************************
C
      HAV2=HAVG**2
C
C     QDELT=(QDEM-QMIND)/(QMAXD-QMIND)
C
      DO 16 I=1,NCOMB
      QSUM=0.0
      CSUM=0.0
C
      QPN=QN(I,1)+(QN(I,2)*HAVG)+(QN(I,3)*HAV2)
      QPM=QM(I,1)+(QM(I,2)*HAVG)+(QM(I,3)*HAV2)
      QPX=QX(I,1)+(QX(I,2)*HAVG)+(QX(I,3)*HAV2)
C
C     SET X'S AND Y'S TO DEMAND AND FLOWRATE
      X0=QMAXD
      Y0=QPX
      X1=QMIDD
      Y1=QPM
      X2=QMIND
      Y2=QPN
C
C     CALL CURVE FITTING ROUTINE
      CALL SCURVE(IBUG)
      QP(I)=C1 + C2*QDEM + C3*QDEM*QDEM
C     QP(I)=QPN+((QPX-QPN)*QDELT)
C
      CPN=CN(I,1)+(CN(I,2)*HAVG)+(CN(I,3)*HAV2)
      CPM=CM(I,1)+(CM(I,2)*HAVG)+(CM(I,3)*HAV2)
      CPX=CX(I,1)+(CX(I,2)*HAVG)+(CX(I,3)*HAV2)
C
C     SET X'S AND Y'S TO DEMAND AND FLOWRATE
      X0=QMAXD
      Y0=CPX
      X1=QMIDD
      Y1=CPM
```

Figure E3. (Sheet 3 of 7)

```
      X2=QMIND
      Y2=CPN
C
C     CALL CURVE FITTING ROUTINE
      CALL SCURVE(IBUG)
      CP(I)=C1 + C2*QDEM + C3*QDEM*QDEM
C
C
C     CP(I)=CPN+((CPX-CPN)*QDELT)
C
      IF(IBUG.GE.1)WRITE(6,99)I,QREQ,QP(I),CP(I)
   99 FORMAT(' I,QREQ, QP(I), CP(I)  ',I5,3F10.2)
C
   16 CONTINUE
C
      IA=0
      IB=1
      QB(1)=0.0
      CB(1)=0.0
      IBP(1)=NCOMB+1
C
C     **********************************
C     * SEPARATE COMBINATIONS INTO TWO *
C     * GROUPS;  THOSE ABOVE AND THOSE *
C     * BELOW THE OPERATING POINT      *
C     **********************************
C
      DO 20 I=1,NCOMB
      IF(QP(I).LE.QREQ)GOTO 18
      IA=IA+1
      IAP(IA)=I
      QA(IA)=QP(I)
      CA(IA)=CP(I)
      GO TO 20
   18 IB=IB+1
      IBP(IB)=I
      QB(IB)=QP(I)
      CB(IB)=CP(I)
   20 CONTINUE
C
      IF(IA.LE.0)GO TO 300
C
C     *************************
C     * ENUMERATE THE POSSIBLE *
C     * COMBINATIONS OF PUMP   *
C     * COMBINATIONS           *
C     *************************
C
      IC=0
C
      DO 24 I=1,IA
      DO 22 J=1,IB
      IC=IC+1
      FB=(QREQ-QA(I))/(QB(J)-QA(I))
```

Figure E3. (Sheet 4 of 7)

```
      FA=(1-FB)
      IAC(I )=IAP(I)
      IBC(IC)=IBP(J)
      QCA(IC)=QA(I)
      QCB(IC)=QB(J)
      FCA(IC)=FA
      FCB(IC)=FB
      CRA(IC)=CA(I)
      CRB(IC)=CB(J)
      CCA(IC)=.0006*DTIME*PKWHR*FA*CA(I)*QA(I)
      CCB(IC)=.0006*DTIME*PKWHR*FB*CB(J)*QB(J)
      COST(IC)=CCA(IC)+CCB(IC)
      IF(IBUG.GE.1)WRITE(6,97)IC,COST(IC)
      IF(IBUG.GE.1)WRITE(6,96)IAC(IC),QCA(IC),FCA(IC),CCA(IC),CRA(IC)
      IF(IBUG.GE.1)WRITE(6,95)IBC(IC),QCB(IC),FCB(IC),CCB(IC),CRB(IC)
   97 FORMAT('IC,COST',I5,F10.2)
   96 FORMAT('IAC,QCA,FCA,CCA,CRA',I5,4F10.2)
   95 FORMAT('IBC,QCB,FCB,CCB,CRB',I5,4F10.2)
   22 CONTINUE
  24  CONTINUE
C
C     *********************
C     * SORT AND RANK THE *
C     * COMBINATIONS      *
C     *********************
C
      JUMP = IC
  26  JUMP=JUMP/2
      IF(JUMP.EQ.0)GO TO 90
      J2=IC-JUMP
      DO 30 J=1,J2
      I=J
   28 J3=I+JUMP
      IF(COST(I).LE.COST(J3))GO TO 30
C
      CALL SWAP(COST(I),COST(J3))
      CALL SWAP(QCA(I),QCA(J3))
      CALL SWAP(QCB(I),QCB(J3))
      CALL SWAP(FCA(I),FCA(J3))
      CALL SWAP(FCB(I),FCB(J3))
      CALL SWAP(CCA(I),CCA(J3))
      CALL SWAP(CCB(I),CCB(J3))
      CALL SWAP(IAC(I),IAC(J3))
      CALL SWAP(IBC(I),IBC(J3))
      CALL SWAP(CRA(I),CRA(J3))
      CALL SWAP(CRB(I),CRB(J3))
C
      I=I-JUMP
      IF(I.GT.0)GO TO 28
   30 CONTINUE
      GOTO 26
C
C     ******************
C     * OUTPUT RESULTS *
```

Figure E3. (Sheet 5 of 7)

```
C      ******************
C
   90 WRITE(6,40)
      WRITE(6,41)HAVG
      WRITE(6,42)QDEM
      WRITE(6,51)QREQ
      WRITE(6,43)PKWHR
      WRITE(6,44)DTIME
C
   40 FORMAT(' PUMP EFFICIENY PROGRAM '/)
   41 FORMAT(' AVG STATIC HEAD        = ',F10.2)
   42 FORMAT(' SYSTEM DEMAND (GPM) = ',F10.2)
   51 FORMAT(' PUMP DEMAND    (GPM) = ',F10.2)
   43 FORMAT(' KILOWATT RATE          = ',F10.2)
   44 FORMAT(' TIME INTERVAL          = ',F10.2/)
C
      DO 50 I=1,IC
      WRITE(6,45)I,COST(I)
   45 FORMAT(//' SOLUTION NUMBER = ',I5,'  TOTAL COST = ',F10.2/)
      J=IAC(I)
      K=IBC(I)
      WRITE(6,46)(NNPC(J,J1),J1=1,6),QCA(I),FCA(I),CRA(I),CCA(I)
      WRITE(6,47)(NNPC(K,K1),K1=1,6),QCB(I),FCB(I),CRB(I),CCB(I)
   46 FORMAT(' PUMPS = ',6I2,' FLOW = ',F8.2,' P = ',F4.2,' C KGAL = ',F
     *6.2,' COST = ',F8.2)
   47 FORMAT(' PUMPS = ',6I2,' FLOW = ',F8.2,' P = ',F4.2,' C KGAL = ',F
     *6.2,' COST = ',F8.2)
   50 CONTINUE
C
      GO TO 500
C
  200 WRITE(6,48)
   48 FORMAT(' SYSTEM DEMANDS SATISFIED BY TANK - NO PUMP NECESSARY')
      GO TO 500
C
  300 WRITE(6,49)
   49 FORMAT(' ALL PUMP CAPACITIES TOO LOW FOR REQUIRED DEMAND')
  500 CONTINUE
C
      END
C
C      *******************
C      * SUBROUTINE SWAP *
C      *******************
C
      SUBROUTINE SWAP(A,B)
C
      REAL A,B,HOLD
C
      HOLD=A
      A=B
      B=HOLD
      RETURN
      END
```

Figure E3. (Sheet 6 of 7)

```
C
C     *********************
C     * SUBROUTINE SCURVE *
C     *********************
C
      SUBROUTINE SCURVE(IBUG)
C
      COMMON/BLK1/X0,Y0,X1,Y1,X2,Y2,C1,C2,C3
C
      REAL X0,Y0,X1,Y1,X2,Y2
      DOUBLE PRECISION R0,R1,R2
C
      IF(IBUG.GE.1)WRITE(6,1)
    1 FORMAT(' ENTER SCURVE ')
      IF(IBUG.GE.1)WRITE(6,2)X0,Y0,X1,Y1,X2,Y2
    2 FORMAT(' X0,Y0,X1,Y1,X2,Y2 ',6F14.6)
      XX0=(X0-X1)*(X0-X2)
      R0=Y0/XX0
      XX1=(X1-X0)*(X1-X2)
      R1=Y1/XX1
      XX2=(X2-X0)*(X2-X1)
      R2=Y2/XX2
C
      C1=R0*X1*X2+R1*X0*X2+R2*X0*X1
      C2=(-R0*(X1+X2))-(R1*(X0+X2))-(R2*(X0+X1))
      C3=R0+R1+R2
C
    3 FORMAT(' EXIT SCURVE ')
      IF(IBUG.GE.1)WRITE(6,4)XX0,XX1,XX2
    4 FORMAT(' XX0,XX1,XX2 ',3F12.6)
      IF(IBUG.GE.1)WRITE(6,5)R0,R1,R2,C1,C2,C3
    5 FORMAT(' R0,R1,R2,C1,C2,C3 ',6F12.6)
      IF(IBUG.GE.1)WRITE(6,3)
C
      RETURN
      END
```

Figure E3. (Sheet 7 of 7)

```
C1         1        2.95   38000.0         11          0
C2 114861.         327.8     327.2     60000.     35000.      10000.
N1  1    0      0     0    0     0     0     0
Q1      335.     19224.     326.5     19795.       318.      20373.
C1      335.      .7674     326.5      .7713       318.       .7792
Q2      335.     16898.     326.5     17574.       318.      18240.
C2      335.      .7816     326.5      .7731       318.       .7681
Q3      335.     15433.     326.5     16032.       318.      16629.
C3      335.     0.8112     326.5     0.7972       318.      0.7858
N1  1    2      0     0    0     0     0     0
Q1      335.     33194.     326.5     34234.       318.      35250.
C1      335.      .7864     326.5      .7785       318.       .7726
Q2      335.     29289.     326.5     30307.       318.      31240.
C2      335.      .8320     326.5      .8186       318.       .8065
Q3      335.     24304.     326.5     25465.       318.      26525.
C3      335.     0.9028     326.5     0.8854       318.      0.8702
N1  1    2      3     0    0     0     0     0
Q1      335.     42368.     326.5     43529.       318.      44651.
C1      335.     0.8470     326.5      .8362       318.       .8260
Q2      335.     35627.     326.5     37036.       318.      38350.
C2      335.     0.9116     326.5     0.8968       318.      0.8839
Q3      335.     27147.     326.5     28679.       318.      30107.
C3      335.     1.0311     326.5     1.0041       318.      0.9816
N1  0    0      0     1    0     0     0     0
Q1      335.     21734.     326.5     21988.       318.      22232.
C1      335.      .6744     326.5      .6654       318.       .6555
Q2      335.     18225.     326.5     18729.       318.      19205.
C2      335.      .7443     326.5      .7380       318.       .7314
Q3      335.     15312.     326.5     15912.       318.      16464.
C3      335.     0.7793     326.5     0.7714       318.      0.7647
N1  1    0      0     1    0     0     0     0
Q1      335.     38937.     326.5     39781.       318.      40622.
C1      335.      .7281     326.5      .7228       318        .7185
Q2      335.     34355.     326.5     35586.       318.      36770.
C2      335.      .7677     326.5      .7592       318        .7523
Q3      335.     29444.     326.5     30599.       318.      31690.
C3      335.     0.8088     326.5     0.7968       318.      0.7866
N1  1    2      0     1    0     0     0     0
Q1      335.     52551.     326.5     53906.       318.      55239.
C1      335.      .7607     326.5      .7515       318.       .7434
Q2      335.     46317.     326.5     47811.       318.      49198.
C2      335.     0.8069     326.5     0.7965       318.      0.7865
Q3      335.     36513.     326.5     38204.       318.      39771.
C3      335.     0.8818     326.5     0.8664       318.      0.8533
N1  1    2      3     1    0     0     0     0
Q1      335.     61584.     326.5     63183.       318.      64656.
C1      335.     0.8068     326.5     0.7971       318.      0.7878
Q2      335.     51647.     326.5     53506.       318.      55245.
C2      335.     0.8688     326.5     0.8571       318.      0.8466
Q3      335.     38518.     326.5     40510.       318.      42373.
C3      335.     0.9816     326.5     0.9593       318.      0.9405
N1  0    0      0     1    2     0     0     0
Q1      335.     33302.     326.5     34228.       318.      35136.
C1      335.      .7625     326.5      .7571       318.       .7502
```

Figure E4. Example input for PCP (Continued)

Q2	335.	27680.	326.5	28562.	318.	29386.		
C2	335.	0.8031	326.5	0.7952	318.	0.7884		
Q3	335.	18954.	326.5	19868.	318.	20731.		
C3	335.	0.9509	326.5	0.9267	318.	0.9063		
N1	1	0	0	1	2	0	0	0
Q1	335.	49515.	326.5	51082.	318.	52656.		
C1	335.	.7715	326.5	.7657	318.	.7610		
Q2	335.	43566.	326.5	45076.	318.	46520.		
C2	335.	0.8014	326.5	0.7921	318.	0.7849		
Q3	335.	32445.	326.5	33870.	318.	35206.		
C3	335.	0.9167	326.5	0.8955	318.	0.8773		
N1	1	2	0	1	2	0	0	0
Q1	335.	63253.	326.5	65354.	318.	67371.		
C1	335.	0.7879	326.5	0.7790	318.	.7715		
Q2	335.	54547.	326.5	56311.	318.	57948.		
C2	335.	0.8327	326.5	0.8221	318.	0.8126		
Q3	335.	38626.	326.5	40517.	318.	42294.		
C3	335.	0.9796	326.5	0.9557	318.	0.9353		
N1	1	2	3	1	2	0	0	0
Q1	335.	71740.	326.5	73933.	318.	76014.		
C1	335.	0.8255	326.5	0.8165	318.	0.8083		
Q2	335.	59195.	326.5	61277.	318.	63206.		
C2	335.	0.8874	326.5	0.8752	318.	0.8647		
Q3	335.	40349.	326.5	42497.	318.	44502.		
C3	335.	1.0748	326.5	1.0445	318.	1.0189		

Figure E4. (Concluded)

```
PUMP COMBINATION PROGRAM

AVG STATIC HEAD (FT)    =     327.50     PUMPS  = (PUMP COMBINATIONS)
SYSTEM DEMAND   (GPM)   =   38271.00     FLOW   = (GPM)
PUMP DEMAND     (GPM)   =   29679.75     P      = (PERCENT OPERATION TIME)
KILOWATT RATE   (C/HR)  =       2.95     U COST = (LENTS/1000 GALLONS/HR)
TIME INTERVAL   (HRS)   =       1.00     COST   = (DOLLARS)


SOLUTION NUMBER =     1                                TOTAL COST =   39.38

PUMPS =  7 0 0 7 0 0 FLOW = 36040.16 P =  .63 U COST =   .76 COST =   30.12
PUMPS =  0 0 0 7 0 0 FLOW = 19075.12 P =  .37 U COST =   .73 COST =    9.26

SOLUTION NUMBER =     2                                TOTAL COST =   39.68

PUMPS =  7 0 0 7 0 0 FLOW = 36040.16 P =  .82 U COST =   .76 COST =   39.68
PUMPS =  0 0 0 0 0 0 FLOW =      .00 P =  .18 U COST =   .00 COST =     .00

SOLUTION NUMBER =     3                                TOTAL COST =   39.86

PUMPS =  7 0 0 7 0 0 FLOW = 36040.16 P =  .65 U COST =   .76 COST =   31.43
PUMPS =  7 0 0 0 0 0 FLOW = 17747.33 P =  .35 U COST =   .77 COST =    8.43

SOLUTION NUMBER =     4                                TOTAL COST =   40.15

PUMPS =  7 0 0 7 8 0 FLOW = 45981.86 P =  .39 U COST =   .79 COST =   25.19
PUMPS =  0 0 0 7 0 0 FLOW = 19075.12 P =  .61 U COST =   .73 COST =   14.96

SOLUTION NUMBER =     5                                TOTAL COST =   40.25

PUMPS =  7 8 0 7 0 0 FLOW = 48640.10 P =  .36 U COST =   .79 COST =   24.41
PUMPS =  0 0 0 7 0 0 FLOW = 19075.12 P =  .64 U COST =   .73 COST =   15.84

SOLUTION NUMBER =     6                                TOTAL COST =   40.70

PUMPS =  7 8 0 7 8 0 FLOW = 57674.90 P =  .27 U COST =   .81 COST =   22.78
PUMPS =  0 0 0 7 0 0 FLOW = 19075.12 P =  .73 U COST =   .73 COST =   17.91

SOLUTION NUMBER =     7                                TOTAL COST =   41.01

PUMPS =  7 0 0 7 8 0 FLOW = 45981.86 P =  .42 U COST =   .79 COST =   27.01
PUMPS =  7 0 0 0 0 0 FLOW = 17747.33 P =  .58 U COST =   .77 COST =   14.00
```

Figure E5. Example output from PCP

APPENDIX F: TOP - TANK OPERATION PROGRAM

1. The tank operation program (TOP) may be used to generate an optimal tank trajectory for the controlling storage tank in a given pump service area. The optimal tank trajectory is a curve that indicates the optimal tank level at a given time during a specified operating period. The controlling storage tank is that tank which controls the hydraulics of the associated pumping station. The tank operation program is based on dynamic programming. A summary of the theoretical basis for the program is provided in Appendix E.

2. Once the optimal tank trajectory has been determined, the optimal pump combinations can be obtained by applying the pump combination program (PCP) to each time interval. To determine the optimal tank trajectory the following data are required: the physical characteristics of the tank, the range and number of allowable tank levels, the number and length of the time intervals in the desired trajectory, the electric rate schedule, the system demand pattern, and a set of three cost operation curves.

3. The cost operation curves are used by the program to determine the cost associated with a particular tank transition (the transition from one water surface elevation to another over a specified period of time) and required pump flow, and are developed by applying the PCP (described in Appendix E) for a range of system demands and tank transitions. Three different cost operation curves are required to describe the system dynamics. The first curve represents the cost associated with a given pump discharge when the tank is filling at a rate of N feet per hour. The second curve represents the cost associated with a given pump discharge when the tank is neither draining nor filling. The third curve represents the cost associated with a given pump discharge when the tank is draining at a rate of N feet per hour.

4. The tank operation program is written in Fortran and is run in a batch mode. Input to the program is read from a user-supplied data file, and output is directed to a user-specified output file. Instructions for data preparation for TOP are given in Table F1. The first card is used to specify the number of time intervals in the operating period and the length of each time interval in hours. The second card is used to specify the average cross-sectional area of the controlling tank, the initial tank level, the maximum tank level, the minimum tank level, and the number of intermediate tank levels. The next three cards are used to specify the three cost operation

Table F1

Data Input Instructions for TOP

Card Group	Format	Column	Description	Variable Name
			TIME DATA	
K1	2X	1-2	Card Group Identifier	
	I8	3-10	Number of Time Intervals	NST
	I10	11-20	Length of Each Time Interval (hr)	IDHR
	I10	20-30	Debug Flag	IBUG
			TANK DATA	
K2	2X	1-2	Card Group Identifier	
	F8.0	3-10	Tank Area (sq ft)	ATANK
	F10.0	11-20	Initial Tank Elevation (ft)	EINT
	F10.0	21-30	Maximum Tank Elevation (ft)	EMAX
	F10.0	31-40	Minimum Tank Elevation (ft)	EMIN
	I10	41-50	Number of Intermediate Elevations	NES
			FIRST COST OPERATION CURVE	
C1	2X	1-2	Card Group Identifier	
	F8.0	3-10	Tank Filling Rate F (ft/hr)	F
	F10.0	11-20	First Flow Rate (gpm)	X0
	F10.0	21-30	First Unit Cost ($/hr)	Y0

(Continued)

(Sheet 1 of 3)

Table F1 (Continued)

Card Group	Format	Column	Description	Variable Name
C1	F10.0	31-40	Second Flow Rate (gpm)	X1
	F10.0	41-50	Second Unit Cost ($/hr)	Y1
	F10.0	51-60	Third Flow Rate (gpm)	X2
	F10.0	61-70	Third Unit Cost ($/hr)	Y2
			SECOND COST OPERATION CURVE	
C2	2X	1-2	Card Group Identifier	
	F8.0	3-10	LEAVE BLANK	
	F10.0	11-20	First Flow Rate (gpm)	X0
	F10.0	21-30	First Unit Cost ($/hr)	Y0
	F10.0	31-40	Second Flow Rate (gpm)	X1
	F10.0	41-50	Second Unit Cost ($/hr)	Y1
	F10.0	51-60	Third Flow Rate (gpm)	X2
	F10.0	61-70	Third Unit Cost ($/hr)	Y2
			THIRD COST OPERATION CURVE	
C3	2X	1-2	Card Group Identifier	
	F8.0	3-10	Tank Draining Rate (ft/hr)	D
	F10.0	11-20	First Flow Rate (gpm)	X0
	F10.0	21-30	First Unit Cost ($/hr)	Y0
	F10.0	31-40	Second Flow Rate (gpm)	X1
	F10.0	41-50	Second Unit Cost ($/hr)	Y1

(Continued)

(Sheet 2 of 3)

Table F1 (Concluded)

Card Group	Format	Column	Description	Variable Name
C3	F10.0	51-60	Third Flow Rate (gpm)	X2
	F10.0	61-70	Third Unit Cost ($/hr)	Y2
			PUMP CONSTRAINT DATA	
Q1	2X	1-2	Card Group Identifier	
	F8.0	3-10	Maximum Pump Discharge (gpm)	QMAX
	F10.0	11-20	Minimum Pump Discharge (gpm)	QMIN
			SYSTEM DEMAND AND ELECTRIC RATE SCHEDULES	
			Repeat for Each Time Interval I, I = 1, NST	
T1	2X	1-2	Card Group Identifier	
	F8.0	3-10	Time (hr)	TYME(I)
	F10.0	11-20	Demand (gpm)	QD(I)
	F10.0	21-30	Electric Rate (¢/kWhr)	RKWD(I)

(Sheet 3 of 3)

curves. Each curve is described using three different data points. Each data point is defined by a unit cost and an associated pump discharge for a given tank transition. The final card is repeated for each time interval in the simulation. For each interval, the time (ending hour), average system demand, and electric rate are specified.

5. A complete listing of the program is provided as Figure F1. A typical input data file is shown in Figure F2. A typical output data file is shown in Figure F3.

```
C                        ********************************
C                        * TOP - TANK OPERATION PROGRAM *
C                        *                              *
C                        * AUTHOR - LINDELL E ORMSBEE   *
C                        *                              *
C                        * LATEST REVISION 12/1/86      *
C                        *                              *
C                        ********************************
C
C     DIMENSION TYME(25),E(25,51),IP(25,51),C3(25,51),CC(25,51),QT(25,51
C    1),QP(25,51)
      DIMENSION TYME(25),E(25,101),IP(25,101),C3(25,101),CC(25,101)
      DIMENSION QD(25),RKWD(25)
C
      COMMON /BLK1/ XO,YO,X1,Y1,X2,Y2
      COMMON /BLK2/ COE1,COE2,COE3
      COMMON /BLK3/ DMAX,DMIN,CX1,CX2,CX3,CM1,CM2,CM3,CN1,CN2,CN3
C
      CHARACTER*14 FILOT,FILIN
C
C     REAL XO,YO,X1,Y1,X2,Y2
C
  201 FORMAT(I10)
  202 FORMAT(F10.0)
C
      WRITE(*,1)
    1 FORMAT('/ INPUT NAME OF INPUT FILE'/)
      READ(*,3)FILIN
      WRITE(*,2)
    2 FORMAT('/ INPUT NAME OF OUTPUT FILE'/)
      READ(*,3)FILOT
    3 FORMAT(A14)
C
      OPEN(5,FILE=FILIN,STATUS='OLD')
      OPEN(6,FILE=FILOT,STATUS='NEW')
C
C     ****************
C     * READ IN DATA *
C     ****************
C
C     NST = NUMBER OF TIME INTERVALS
C     IDHR = LENGTH OF EACH TIME INTERVAL (HRS)
C     IBUG = DEBUG FLAG (0 = NORMAL OUTPUT, 1 = EXTENDED OUTPUT)
C
      READ(5,4)NST,IDHR,IBUG
    4 FORMAT(2X,I8,2I10)
C
C     ATANK = TANK AREA (FT2)
C     EINT = INITIAL ELEV (FT)
C     EMAX = MAXIMUM ELEVATION OF TANK
C     EMIN = MINIMUM ELEVATION OF TANK
C     NES = NUMBER OF INTERMEDIATE ELEVATIONS
C
      READ(5,5)ATANK,EINT,EMAX,EMIN,NES
```

Figure F1. Program listing for TOP (Sheet 1 of 8)

```
    5   FORMAT(2X,F8.0,3F10.0,I1)
C
        COE1=0.0
        COE2=0.0
        COE3=0.0
C
        KP1=0
        QP1=0.0
        PT1=0.0
C
C       READ IN DISCHARGE VS UNIT COST CURVES FOR (+,0,-) DELTA H CURVES
C
C               X = Q, Y = C
C
C       DMAX = MAX DELTA H
C       DMIN = MIN DELTA H
C
    8   FORMAT(2X,F8.0,6F10.0)
C
        READ(5,8)DMAX,X0,Y0,X1,Y1,X2,Y2
        DMAX=DMAX*IDHR
        X0=X0/10000.0
        X1=X1/10000.0
        X2=X2/10000.0
        CALL SCURVE(IBUG)
        CX1=COE1
        CX2=COE2
        CX3=COE3
        READ(5,8)DDUM,X0,Y0,X1,Y1,X2,Y2
        X0=X0/10000.0
        X1=X1/10000.0
        X2=X2/10000.0
        CALL SCURVE(IBUG)
        CM1=COE1
        CM2=COE2
        CM3=COE3
        READ(5,8)DMIN,X0,Y0,X1,Y1,X2,Y2
        DMIN=DMIN*IDHR
        X0=X0/10000.0
        X1=X1/10000.0
        X2=X2/10000.0
        CALL SCURVE(IBUG)
        CN1=COE1
        CN2=COE2
        CN3=COE3
C
C       READ IN PUMP CONSTRAINTS
C
        READ(5,9)QMAX,QMIN
    9   FORMAT(2X,F8.0,F10.0)
C
C       READ IN DEMAND AND RATE SCHEDULES
C
        DO 12 I=1,NST
```

Figure F1. (Sheet 2 of 8)

```
      READ(5,11)TYME(I),QD(I),RKWD(I)
 11   FORMAT(2X,F8.0,2F10.0)
 12   CONTINUE
C
C     ************************
C     * GENERATE STATE SPACE *
C     ************************
C
      DEE=(EMAX-EMIN)/(NES-1)
C
      DO 40 K=1,NES
C
      EINC=EMIN+((K-1)*DEE)
C
      DO 30 I=1,NST
C
      E(I,K)=EINC
C
  30  CONTINUE
C
  40  CONTINUE
C
      TANKC=ATANK/(IDHR*3600)
C
C     ************************
C     * VARIABLE DESCRIPTION *
C     ************************
C
C     IP(I,K) = OPTIMAL STATE UPSTREAM FROM STATE K
C     QT(I,K) = TANK FLOW
C     C3(I,K) = COST FOR TRANSITION ENDING IN STATE K
C     CC(I,K) = CUMMULATIVE COST FOR STATE K
C     QP(I,K) = PUMP DISCHARGE FOR STAGE I STATE K
C
C     **************************
C     * EVALUATE INITIAL STAGE *
C     **************************
C
      IFLAG=0
C
      DO 60 K=1,NES
      C3(1,K)=9999.0
      CC(1,K)=9999.0
C
      DE=E(1,K)-EINT
      IF(ABS(DE).GT.DMAX.OR.ABS(DE).GT.DMIN)GO TO 45
C
      QTT=DE*TANKC*448.8
      QR=QD(1)+QTT
      ISI=1
C
      HAVG=(E(1,K)+EINT)/2.0
      ERATE=RKWD(1)
C
```

Figure F1. (Sheet 3 of 8)

```
      QDEM=QD(1)
C
      IF(IBUG.GE.1)WRITE(6,51)ISI,ISI,K,HAVG,QD(1),QTT,QR
   51 FORMAT(/' STAGE = ',I5,' DEC = ',2I5,' H,QD,QT,QR ',4F12.2/)
C
      IF(QR.GT.QMIN.AND.QR.LT.QMAX)GO TO 50
      IF(IBUG.GE.1)WRITE(6,52)
   52 FORMAT(' DECISION INFEASIBLE DUE TO QR OUTSIDE BOUNDS')
      GO TO 60
C
   45 IF(IBUG.GE.2)WRITE(6,652)
  652 FORMAT(' DECISION INFEASIBLE DUE TO DE OUTSIDE BOUNDS')
      GO TO 60
C
   50 CALL COST(IBUG,DE,ICHR,QR,QDEM,ERATE,OCST)
C
      IF(IBUG.GE.1)WRITE(6,63)ISI,ISI,K,OCST
   63 FORMAT(' STAGE = ',I5,' DEC = ',2I5,' COST = ',F10.2)
C
      IF(OCST.LE.9999)IFLAG=1
C
      C3(1,K)=OCST
      CC(1,K)=OCST
      IP(1,K)=1
   60 CONTINUE
C
      IF(IFLAG.LE.0)WRITE(6,62)
   62 FORMAT(' INITAL STAGE INFEASIBLE')
      IF(IFLAG.LE.0)GO TO 120
C
C     ****************************
C     * EVALUATE REMAINING STAGES *
C     ****************************
C
      DO 90 I=2,NST
C
      J=I-1
C
      DO 80 K=1,NES
C
      SCST=9999.0
      IP(I,K)=1
      C3(I,K)=9999.0
      CC(I,K)=9999.0
C
      DO 70 L=1,NES
C
C     IF UPSTREAM STATE INFEASIBLE GO TO NEXT STATE
C
      IF(IBUG.GE.1.AND.CC(J,L).GE.9999.0)WRITE(6,71)I,L,K,L
   71 FORMAT(' STAGE ',I5,' DEC ',2I5,' DEC ',I5,' INFEASIBLE')
      IF(CC(J,L).GE.9999.0)GO TO 70
C
C     DETERMINE CHANGE IN TANK LEVEL
```

Figure F1. (Sheet 4 of 8)

```
C
      DE=E(I,K)-E(J,L)
      IF(ABS(DE).GT.DMAX.OR.ABS(DE).GT.DMIN)GO TO 75
C
C     DETERMINE REQUIRED FLOWRATE
C
      QTT=DE*TANKC*448.8
      QR=QD(I)+QTT
C
      HAVG=(E(I,K)+E(J,L))/2.0
      ERATE=RKWD(I)
C
      QDEM=QD(I)
C
      IF(IBUG.GE.1)WRITE(6,73)I,L,K,HAVG,QDEM,QTT,QR
   73 FORMAT(/' STAGE = ',I5,' DEC = ',2I5,' H,QD,QT,QR ',4F10.2)
C
C     IF QR<0 TANK FLOW EXCEEDS SYSTEM DEMAND
C     AND DECISION IS INFEASIBLE - GO TO NEXT STATE
C
      IF(QR.GT.QMIN.AND.QR.LT.QMAX)GO TO 65
C
      IF(IBUG.GE.1)WRITE(6,72)
   72 FORMAT(' DECISION INFEASIBLE DUE TO QR OUTSIDE BOUNDS')
      GO TO 70
C
   75 IF(IBUG.GE.2)WRITE(6,772)
  772 FORMAT(' DECISION INFEASIBLE DUE TO DE OUTSIDE BOUNDS')
      GO TO 70
C
   65 CALL COST(IBUG,DE,IDHP,QR,QDEM,ERATE,OCST)
C
C     DETERMINE CUMMULATIVE COST - CCOST
C
      CCOST=OCST+CC(J,L)
C
      IF(IBUG.GE.1)WRITE(6,74)I,L,K,OCST,CCOST
   74 FORMAT(' STAGE = ',I5,' DEC = ',2I5,' C,CC ',2F10.2)
C
C     IF NEW TRANSITION IS BETTER THAN OLD ONE UPDATE VARIABLES
C
      IF(SCST.LT.CCOST)GO TO 70
C
C     UPDATE BEST CUMMULATIVE COST
C
      IP(I,K)=L
C     QT(I,K)=QTT
      SCST=CCOST
      C3(I,K)=OCST
      CC(I,K)=SCST
C
   70 CONTINUE
   80 CONTINUE
   90 CONTINUE
```

Figure F1. (Sheet 5 of 8)

```
C
      DO 110 K=1,NES
      WRITE(6,13)EINT,E(NST,K),CC(NST,K)
   13 FORMAT(/'INITIAL E = ',F10.2,'  FINAL E = ',F10.2,'  TOTAL COST
     1= ',F10.2/)
C
      IF(CC(NST,K).LT.9999)GO TO 95
      WRITE(6,14)K
   14 FORMAT(/'STATE',I5,' IS INFEASIBLE'/)
      GO TO 110
C
   95 WRITE(6,15)
C
      L=K
C
C     ****************************
C     *  DETERMINE OPTIMAL PATH  *
C     ****************************
C
      DO 100 J=1,NST
C
      I=NST+1-J
      LL=IP(I,L)
      IF(LL.LE.0)WRITE(6,18)I,L
      IF(LL.LE.0)GO TO 100
C
      QTTT=(E(I,L)-EINT)*TANKC*448.8
      IF(I.NE.1)QTTT=(E(I,L)-E(I-1,LL))*TANKC*448.8
      QPPP=QD(I)+QTTT
C
C     ******************
C     * OUTPUT RESULTS *
C     ******************
C
   15 FORMAT(/' I J K   TIME    RATE     E(K)       QD(I)       QT(I)
     *  QP(I)       COST'/)
      WRITE(6,16)I,LL,L,TYME(I),RKWD(I),E(I,L),QD(I),QTTT,QPPP,C3(I,L)
   16 FORMAT(3I2,F8.2,F8.2,F8.2,3F12.2,F10.2)
   17 FORMAT(3I2,3F8.2)
   18 FORMAT(' ALL STATES INFEASIBLE FOR STAGE = ',I5,' STATE = ',I5/)
C
      L=LL
C
  100 CONTINUE
  110 CONTINUE
  120 CONTINUE
C
      CLOSE(5)
      CLOSE(6)
      END
C
C     *********************
C     * SUBROUTINE SCURVE *
C     *********************
```

Figure F1. (Sheet 6 of 8)

```
C
C     THIS SUBROUTINE FITS A QUADRATIC CURVE THROUGH
C     THREE SUPPLIED POINTS USING LAGRANGIAN POLYNOMIALS
C
      SUBROUTINE SCURVE(IBUG)
C
      COMMON /BLK1/ X0,Y0,X1,Y1.X2,Y2
      COMMON /BLK2/ COE1,COE2,COE3
C
      REAL X0,Y0,X1,Y1,X2,Y2
      DOUBLE PRECISION R0,R1,R2
C
      XX0=(X0-X1)*(X0-X2)
      R0=Y0/XX0
      XX1=(X1-X0)*(X1-X2)
      R1=Y1/XX1
      XX2=(X2-X0)*(X2-X1)
      R2=Y2/XX2
C
      COE1=R0*X1*X2+R1*X0*X2+R2*X0*X1
      COE2=(-R0*(X1+X2))-(R1*(X0+X2))-(R2*(X0+X1))
      COE3=R0+R1+R2
C
      IF(IBUG.GE.2)WRITE(6,10)COE1,COE2,COE3
   10 FORMAT(/' C0,C1,C2 ',3F10.6/)
C
      RETURN
      END
C
C     *******************
C     * SUBROUTINE COST *
C     *******************
C
C     THIS SUBROUTINE DETERMINES THE COST  TO
C     MOVE   FROM ONE STATE (ELEV) TO ANOTHER
C     STATE (ELEV)  BY INTERPOLATING  BETWEEN
C     THE THREE COST OPERATION CURVES
C
      SUBROUTINE COST(IBUG,DE,IDHR,QREQ,QDEM,ERATE,OCST)
C
      COMMON /BLK1/ X0,Y0,X1,Y1,X2,Y2
      COMMON /BLK2/ COE1,COE2,COE3
      COMMON /BLK3/ DMAX,DMIN,CX1,CX2,CX3,CM1,CM2,CM3,CN1,CN2,CN3
C
      OCST=999999.0
C
      RREQ=QREQ/10000.0
      RREQ2=RREQ*RREQ
C
      CPX=CX1+(CX2*RREQ)+(CX3*RREQ2)
      CPM=CM1+(CM2*RREQ)+(CM3*RREQ2)
      CPN=CN1+(CN2*RREQ)+(CN3*RREQ2)
C
      IF(DE.LE.0.0)GO TO 10
```

Figure F1. (Sheet 7 of 8)

```
C
      OCST=CPM+((CPX-CPM)*DE/DMAX)
      GO TO 20
C
   10 OCST=CPM+((CPM-CPN)*DE/DMIN)
C
   20 OCST=OCST*IDHR*ERATE
      IF(IBUG.GE.1)WRITE(6,30)DMAX,DMIN,IDHR,ERATE
   30 FORMAT(' X,N,T,R ',4F10.2)
      IF(IBUG.GE.1)WRITE(6,40)DE,CPX,CPM,CPN,OCST
   40 FORMAT(' DE,CPX,M,N,OCST ',5F10.2)
C
      RETURN
      END
```

Figure F1. (Sheet 8 of 8)

```
K1        24         1         0
K2114861.0    327.80     334.0     326.0       101
C1       1.0    20000.      9.56    30000.    14.350     40000    19.7500
C2              10000.      4.65    35000.    16.020    60000.    27.8200
C3       1.0    10000.      4.49    35000.    15.200    60000.    27.1900
Q1    10000.    60000.
T1       1.0   38271.0      2.95
T1       2.0   38827.0      2.95
T1       3.0   17700.0      2.95
T1       4.0    1311.0      2.95
T1       5.0   15625.0      2.95
T1       6.0   18580.0      2.95
T1       7.0   21358.0      2.95
T1       8.0   27500.0      2.95
T1       9.0   30710.0      2.95
T1      10.0   32141.0      2.95
T1      11.0   33850.0      2.95
T1      12.0   36922.0      2.95
T1      13.0   33850.0      2.95
T1      14.0   32766.0      2.95
T1      15.0   31953.0      2.95
T1      16.0   30286.0      2.95
T1      17.0   35347.0      2.95
T1      18.0   38457.0      2.95
T1      19.0   27276.0      2.95
T1      20.0   29498.0      2.95
T1      21.0   29151.0      2.95
T1      22.0   35386.0      2.95
T1      23.0   18349.0      2.95
T1      24.0   16096.0      2.95
```

Figure F2. Example input for TOP

TOTAL COST (DOLLARS) = 908.59

INTIAL ELEVATION (FT) = 327.80

FINAL ELEVATION (FT) = 328.08

I = STAGE INDEX
J = PREVIOUS STATE INDEX
K = CURRENT STATE INDEX
TIME = (HRS)
RATE = (CENTS/HR)
E(K) = ELEVATION ASSOCIATED WITH STATE K (FT)
QD(I) = SYSTEM DEMAND ASSOCIATED WITH STAGE I (GPM)
QT(I) = TANK FLOW (+ INFLOW, - OUTFLOW) (GPM)
QP(I) = PUMP FLOW (GPM)
COST = (DOLLARS)

I J K	TIME	RATE	E(K)	QD(I)	QT(I)	QP(I)	COST
242327	24.00	2.95	328.08	16096.00	4581.86	20677.86	28.22
232023	23.00	2.95	327.76	18349.00	3436.94	21785.94	29.64
222620	22.00	2.95	327.52	35386.00	-6873.44	28512.56	37.74
212926	21.00	2.95	328.00	29151.00	-3436.50	25714.50	34.33
201629	20.00	2.95	328.24	29498.00	14891.80	44389.80	61.20
19 116	19.00	2.95	327.20	27276.00	17183.38	44459.38	61.44
18 8 1	18.00	2.95	326.00	38457.00	-8018.79	30438.21	40.20
1714 8	17.00	2.95	326.56	35347.00	-6873.44	28473.56	37.69
161614	16.00	2.95	327.04	30286.00	-2291.15	27994.85	37.51
151916	15.00	2.95	327.20	31953.00	-3436.50	28516.50	38.09
142319	14.00	2.95	327.44	32766.00	-4582.29	28183.71	37.53
132823	13.00	2.95	327.76	33850.00	-5727.65	28122.35	37.34
123428	12.00	2.95	328.16	36922.00	-6873.44	30048.56	39.80
113934	11.00	2.95	328.64	33850.00	-5727.65	28122.35	37.34
104339	10.00	2.95	329.04	32141.00	-4581.86	27559.14	36.70
94643	9.00	2.95	329.36	30710.00	-3436.94	27273.06	36.42
83146	8.00	2.95	329.60	27500.00	17183.38	44683.38	61.73
72931	7.00	2.95	328.40	21358.00	2291.15	23649.15	32.07
62629	6.00	2.95	328.24	18580.00	3436.50	22016.50	29.96
52226	5.00	2.95	328.00	15625.00	4582.29	20207.29	27.58
41222	4.00	2.95	327.68	1311.00	11455.30	12766.30	17.47
3 812	3.00	2.95	326.88	17700.00	4582.29	22282.29	30.41
216 8	2.00	2.95	326.56	38827.00	-9164.59	29662.41	39.05
1 116	1.00	2.95	327.20	38271.00	-8591.25	29679.75	39.13

Figure F3. Example output from TOP

APPENDIX G: TIME-METERED GENERAL SERVICE SCHEDULE "GT"

1. This appendix presents the Potomac Electric Power Company electric rate schedule* applicable to the Dalecarlia and Bryant Street pumping stations.

Availability - Shall be applicable in the District of Columbia portion of the Company's service area to customers whose maximum 30-minute demand equals or exceeds 1000 kw during two or more billing months per year. Any customer presently on Schedule DC-GT whose maximum 30-minute demand is less than 900 kw for twelve consecutive billing months in a calendar year may at the customer's option elect to continue service on this schedule or elect to be served under any other applicable schedule.

Available for low voltage electric service.

Available for auxiliary or emergency service when modified by Rider No. "GT-2," for primary service when modified by Rider No. "GT-3A" or Rider No. "GT-3B," and for heating service when modified by Rider No. "GT-4."

Not available for temporary service, supplementary loads metered separately from lighting and other usage in the same occupancy, or railway propulsion service.

Character of Service -

Secondary Service - The service supplied under this schedule will be alternating current, sixty hertz, normally three phase, four wire, 120/208 volts or 265/460 volts.

Primary and High Voltage Service - The service under this schedule, when modified by Primary Service Rider "GT-3A," normally will be alternating current, sixty hertz, three phase, three wire, at

---

* Date of Issue: April 2, 1985; Date Effective: April 2, 1985. Issued by William F. Schmidt, Vice President, 1900 Pennsylvania Avenue, NW, Washington, DC 20068.

13.2 KV or 33 KV, and when modified by High Voltage Rider "GT-38," will be 69 kv or above. Primary service voltage levels will be specified by the Company on the basis of its available facilities and the magnitude of the load to be served.

Schedule of Monthly Charges -

	Summer	Winter
A. Customer Charge	$285.00 per month	$285.00 per month
B. Energy Charge		
On-Peak Period	$0.06235 per kwhr	$0.05226 per kwhr
Intermediate Period	$0.04618 per kwhr	$0.04617 per kwhr
Off-Peak Period	$0.02947 per kwhr	$0.02947 per kwhr
C. Production and Transmission Charge		
On-Peak Billing Demand	$9.80 per kw	
D. Distribution Charge	$6.00 per kw	$6.00 per kw
E. Minimum Charge - the Customer Charge and the Distribution Charge		

Season Designation - Summer months, for purposes of application of this rate schedule, are the billing months of June through September; winter months are the billing months of January through May, plus October through December.

Rating Periods -

Weekdays (Excluding Holidays)

On-Peak Period	12:00 noon	to	8:00 p.m.
Intermediate Period	8:00 a.m.	to	12:00 noon
	8:00 p.m.	to	12:00 midnight
Off-Peak Period	12:00 midnight	to	8:00 a.m.

Saturdays, Sundays, and Holidays

Off-Peak Period All Hours

Holidays

New Year's Day, Rev. Martin Luther King's Birthday, Washington's Birthday, Memorial Day, Independence Day, Labor Day, Columbus Day, Veterans' Day, Thanksgiving Day, Christmas Day.

Billing Demands -

Production and Transmission (Summer Months Only) - The billing demand shall be the maximum 30-minute demand recorded during the on-peak period of the billing month.

Distribution (All Months) - The billing demand shall be the maximum 30-minute demand recorded during the billing month, but shall not be less than the highest such demand established during the previous eleven months, except as modified by Rider No. "GT-4."

Fuel Adjustment Charge - The rates stated above include a base fuel cost component of $0.0231985 per kilowatt-hour for secondary service and $0.0224190 per kilowatt-hour for primary and high-voltage service including adjustment for losses. Incremental charges for fuel and interchange, computed in accordance with the provisions of "Fuel Adjustment Charge Rider FA," combined with monthly charges under the provisions of this schedule, constitute the total charge for the services which the Company furnishes.

Meter Reading - Watt-hour meters will be read to the nearest multiple of the meter constant and bills rendered accordingly.

END
DATE
FILMED
MARCH
1988
DTIC